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Technical Report

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16. ABSTRACT

This report details the results of a recently completed five-year Combined Sewer Overflow (CSO) control program in Saginaw, Michigan (10,000 acres, pop. 85,000) funded through the U.S. EPA 108 Great Lakes Demonstration Program and conducted by the engineering contractor, Environmental Design and Planning, Inc. (EDP), Hanover, Massachusetts. The implemented control program entailed modification of 12 combined sewer regulation chambers together with construction of one new in-line control chamber to maximize transient system storage of wet weather combined sewerage for later bleedback to a well-operated AWT/WWTP having phosphorus removal and ample treatment capacity. These improvements, "the partial BMP plan", represent a partial completion of the first of two phases of the City's CSO Facility Plan. The objective of Phase I was to maximize WWTP processing of wet weather combined sewage generated using inexpensive transient system storage (less than $1/cu.ft.) so as to minimize the extent, scale and cost of satellite CSO treatment facilities (Phase II). The swirl concentrator technology was recommended to treat residual overflows remaining after Phase I improvements. Six major Facilities were recommended and adopted as part of the CSO Facility Plan.

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IN-SYSTEM STORAGE CONTROLS FOR REDUCTION OF COMBINED SEWER OVERFLOW-SAGINAW, MICHIGAN

TECHNICAL REPORT

by

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for

DEPARTMENT OF PUBLIC UTILITIES
CITY OF SAGINAW
SAGINAW, MICHIGAN

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DISCLAIMER

This report has been reviewed by the Great Lakes National Program Office and the Municipal Environmental Research Laboratory, U. S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U. S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The Environmental Protection Agency was created because of increasing public and government concerns about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and demonstration is that necessary first step in problem solving and it involves defining the problem, measuring its impact, and searching for solutions. This publication is one of the products of that research, a most vital communications link between the researcher and the user community.

In the implementation of PL92-500 Congress established, through Section 108, demonstration funds for developing realistic and innovative management approaches to solve combined and storm sewer discharges impacting the Great Lakes. This project has been supported by Section 108 funds through the Great Lakes National Programs Office, Region V.

The deleterious effects of storm sewer discharges and combined sewer overflows upon the nation's waterways have become of increasing concern in recent times. Efforts to alleviate the problem depend in part upon the utilization of improved urban runoff control strategies.

This report presents the results of a five-year implementation program effort for management of combined sewer overflows from the City of Saginaw impacting the Saginaw River. A CSO Facility Plan was prepared entailing advanced system management concepts for creating in-system transient storage for later bleedback to a well-operated advanced wastewater treatment plant coupled with state-of-the-art swirl solids concentrator complexes for handling residual overflows. A portion of the system storage management plan was implemented in 1984 and was field evaluated. Roughly ten percent of total wet weather related phosphorous loadings to the Saginaw River was reduced by this program. In addition, total wet weather related suspended solids and biochemical oxygen demand discharging to the Saginaw River were reduced by sixteen percent and twenty percent, respectively, as a direct consequence of this implementation.

Peter Wise, Director
Great Lakes National Program
Office
Chicago, Illinois

ABSTRACT

This report details the results of a recently completed five-year combined sewer overflow (CSO) control program in Saginaw, Michigan (10,000 acres, pop. 85,000) funded through the U.S. EPA 108 Great Lakes Demonstration Program and conducted by the engineering contractor, Environmental Design and Planning, Inc. (EDP), Hanover, Mass. The implemented control program entailed modification of 12 combined sewer regulation chambers together with construction of one new in-line control chamber to maximize transient system storage of wet weather combined sewerage for later bleedback to a well-operated AWT/WWTP having phosphorous removal and ample treatment capacity. These improvements, "the partial BMP plan", represent a partial completion of the first of two phases of the City's CSO Facility Plan. The objective of Phase 1 was to maximize WWTP processing of wet weather combined sewage generated using inexpensive transient system storage (less than $1/cu.ft.) so as to minimize the extent, scale and cost of satellite CSO treatment facilities (Phase II). The swirl concentrator technology was recommended to treat residual overflows remaining after Phase I improvements. Six major Facilities were recommended and adopted as part of the CSO Facility Plan.

EDP determined that city-wide availability of in-line storage without threatening basement flooding was extensive, a total of 3.9 million cu.ft. Regulation chamber internal weirs were modified in one portion of the City to enhance in-line storage and the first of fourteen new in-line storage chambers, (upstream of regulators) was also constructed. New innovative vortex valve flow throttling devices were used to replace existing malfunctioning mechanical float-operated control valves at regulation chambers and another installed at the new in-line control structure. These devices actualized the notion of using a battery of system-wide flow controllers within the hostle environment of a sewer system to reliably and accurately throttle flows, pass debris, create storage and allow self-scouring on draindown without mechanical or electric controls or human interface.

The total cost of the implemented program including engineering and construction was approximately $516,000. A post construction evaluation measurement/modeling program was conducted by EDP in the Fall, 1984 to verify preproject design conditions. Implementation of the reduced BMP plan is estimated to have increased the percentage of total annual wet weather flow directed to the WWTP by 14.6% (from a previous "as is" level of 37.7%) and is estimated to have incrementally reduced (beyond "as is" levels) suspended solids loadings to the Saginaw River by

16%, BOD by 20% and total phosphorous by 8.5% (secondary treatment). During periods of AWT treatment (pickling liquor addition) at the WWTP, total phosphorous removals are estimated to increase to 12.6% above pre-implementation levels. Implementation of the remaining elements of the complete BMP plan will produce an overall incremental wet weather flow increase of 30% (increasing the total wet weather flow treated at the WWTP to 67.7%).

In sum, expenditures of approximately $516,000 under the reduced BMP plan for low level sewerage system modifications (entailing a significant portion of proposed controls for one-half of the City) resulted in decreasing the system-wide total phosphorous wet weather loadings to the river by about 10%. No adverse conditions resulted from the plan. The concept of inexpensive system controls to maximize transient in-line storage of combined sewage with bleedback to a high performance WWTP worked well and is recommended as a management practice.

This project was funded by U.S. EPA in part, to demonstrate new innovative full scale approaches for mitigating CSO impacts on the Great Lakes. The primary project justification was to develop and document an innovative, inexpensive and cost-effective management approach for mitigating wet weather phosphorous loadings to Saginaw Bay (Lake Huron). The results are currently being used as part of U.S. EPA´s joint efforts with Canada to develop phosphorous limitation management strategies. In addition, the results of this demonstration project will provide valuable insights for a number of projects in the formative preliminary phase investigating in-system storage management to "clip" or eliminate CSO from small intensity, high frequency events as a first "stopgap" and inexpensive control measure.

This report was submitted in partial fulfillment of Grant No. S005359 by Environmental Design & Planning, Inc. sponsored by a Section 108 grant from the Great Lakes National Program Office, Region V, U.S. EPA with funds also from the City of Saginaw. This report covers a period of September, 1979 to December, 1984 and work was completed as of April, 1985.

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ABBREVIATIONS AND SYMBOLS

Abbreviations

ac	acre
cos	cosine
cf	cubic feet
cfs	cubic feet per second
cm	centimenter
dwf	dry weather flow
fpm	feet per minute
fps	feet per second
ft	feet
ft^2	square feet
g	gram
gal	gallon
gpm	gallon per minute
h.	hour
imp.	impervious
in.	inch
kg	kilogram
L	liter
lb	pound
m	meter
mg/l	milligram per liter
ml	milliliter
MG	million gallons
sq	square
SWMM	Storm Water Management Model (U.S. EPA)

Symbols

A	Angle of lateral entry at junction
BOD	Biochemical Oxygen Demand (5 days)
CBOD	Carbonacous Biochemical Oxygen Demand
CSO	Combined Sewer Overflow
DO	Dissolved Oxygen
D_R	Ratio of pipe diameters at junction
h	Depth of flow above regulator orifice
g	Acceleration of gravity
H	Head loss due to junction
I	Percent Imperviousness
K	Energy loss coefficient
NBOD	Nitrogenous Biochemical Oxygen Demand
Q_R	Ratio of flows at junction
TSS	Total Suspended Solids
WWTP	Waste Water Treatment Plant
Z	Runoff coefficient
%	Percent

ACKNOWLEDGEMENTS

The authors would like to commend the EDP staff who worked so hard to make this program a success. Through the diligent efforts of many, a study has been prepared assessing the viability of a relatively new technology for solving a complex combined sewer management problem in the City of Saginaw.

We would like to thank the City of Saginaw Public Utilities Department for their extreme cooperation and good will, including Robert Dust, Executive Director of Public Utilities, Mr. Jim Anderson, Superintendent and Fred Schulz, Assistant Superintendent, Wastewater Treatment Division.

We would like to also thank Mr. Ralph G. Christensen, U.S. EPA Region V, Great Lakes National Program Office, for his support and assistance. The Great Lakes Program provided funds for this project. In addition, Mr. Richard P. Traver, U.S. EPA, Edison, New Jersey was extremely helpful in coordinating the early phases of the project along with Mr. Doug Ammon and Mr. John English U.S. EPA, Cincinnati, Ohio.

Finally, we would like to express our sincerest appreciation to Ms. Eunice Harris for her patience in typing this report.

Environmental Design and Planning, Inc. was the engineering contractor for his project. Dr. William C. Pisano, P.E., President, EDP, was the principal investigator. Mr. Daniel J. Connick, Sr. Design Engineer, was the project manager. Mr. Gerald L. Aronson, Executive Vice President, EDP, supervised field monitoring and evaluation efforts.

CHAPTER 1

PROJECT DESCRIPTION

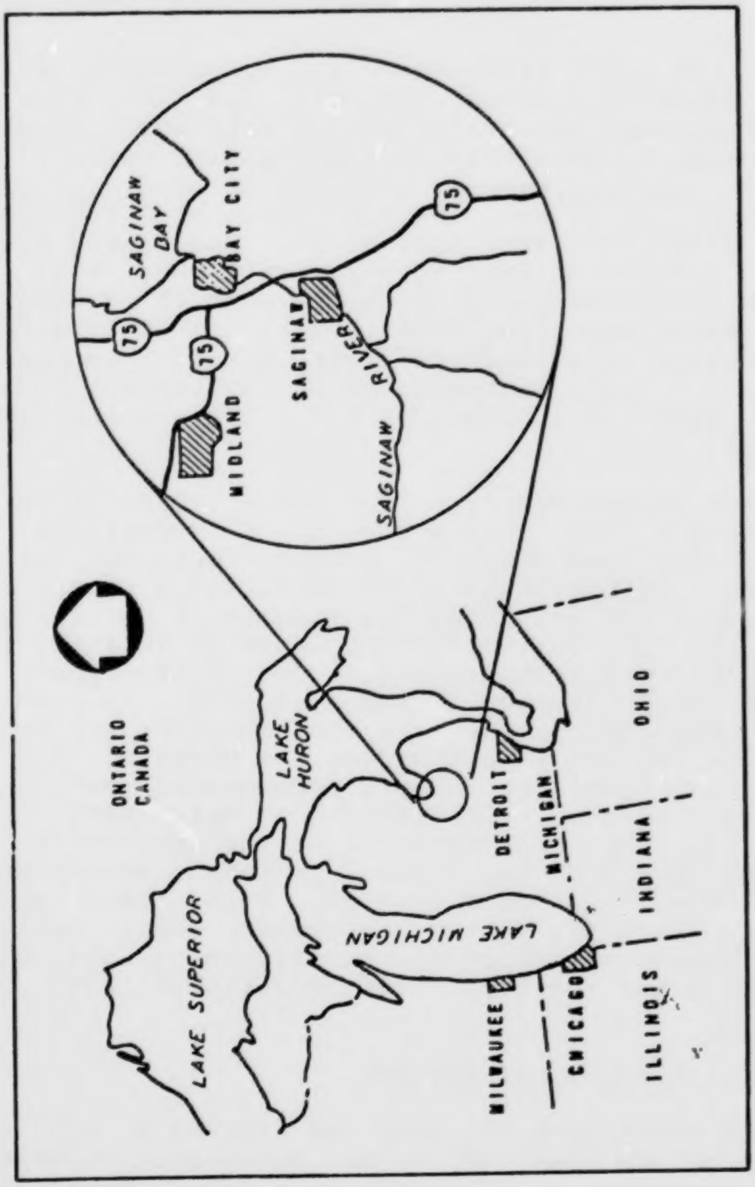
1.1 Background

In 1979 the City of Saginaw in cooperation with the 108 Great Lakes National Program, Region V, U.S. EPA embarked on a program to implement and evaluate on a full-scale basis new innovative low-cost technologies for mitigating combined sewer overflows (CSO) discharging from the City of Saginaw sewerage system into the Saginaw River. The Saginaw River discharges into Saginaw Bay and ultimately into Lake Huron. See Figure 1.1. The 108 Program and subsequent funding was established when Public Law 92-500 came into existence for the purpose of demonstrating on full scale basis promising technologies for abating CSO and stormwater pollution impacting the Great Lakes. Approximately $1.1 million was allocated to the project. Environmental Design & Planning, Inc. (EDP) was chosen by U.S. EPA and the City to perform this work.

The project progressed from 1979 until 1984, culminating with implementation and evaluation of a number of system improvements. The over[illegible] work effort consisted of three separate phases. Within Phase 1 EDP submitted a report to the City of Saginaw which presented the results of a feasibility study in which the most cost effective means of treating CSO in the City of Saginaw were established. During this phase a CSO Facility Plan was developed and adopted as well as recommendations for implementating a demonstration project. In 1983 (Phase 2) a further investigation was prepared due to a lack of funds to implement the Phase 1 recommended demonstration program. Phase 2 efforts were directed at optimizing a reduced scope of work plan for implementation using available funds. Following implementation of the Phase 2 proposed improvements, a Phase 3 evaluation was performed in 1984. This effort generated a greater data base for system hydraulic model calibration allowing for improved estimates of the effects of plan elements completed to date and the effects of additional plan element implementation.

1.2 Chronological Overview of Project

In the early 70´s the City had adopted a CSO control program entailing the use of seven sedimentation/detention/ chlorination facilities. In the mid 70´s one such facility was constructed at the Hancock Street Pumping Station and regulators in the area were modified to enhance flow to this treatment facility. See Figure 1.2 for the general location of this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CHICAGO
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INDIANA
MICHIGAN
DETROIT
OHIO
SAGINAW BAY
BAY CITY
MIDLAND
SAGINAW
SAGINAW RIVER
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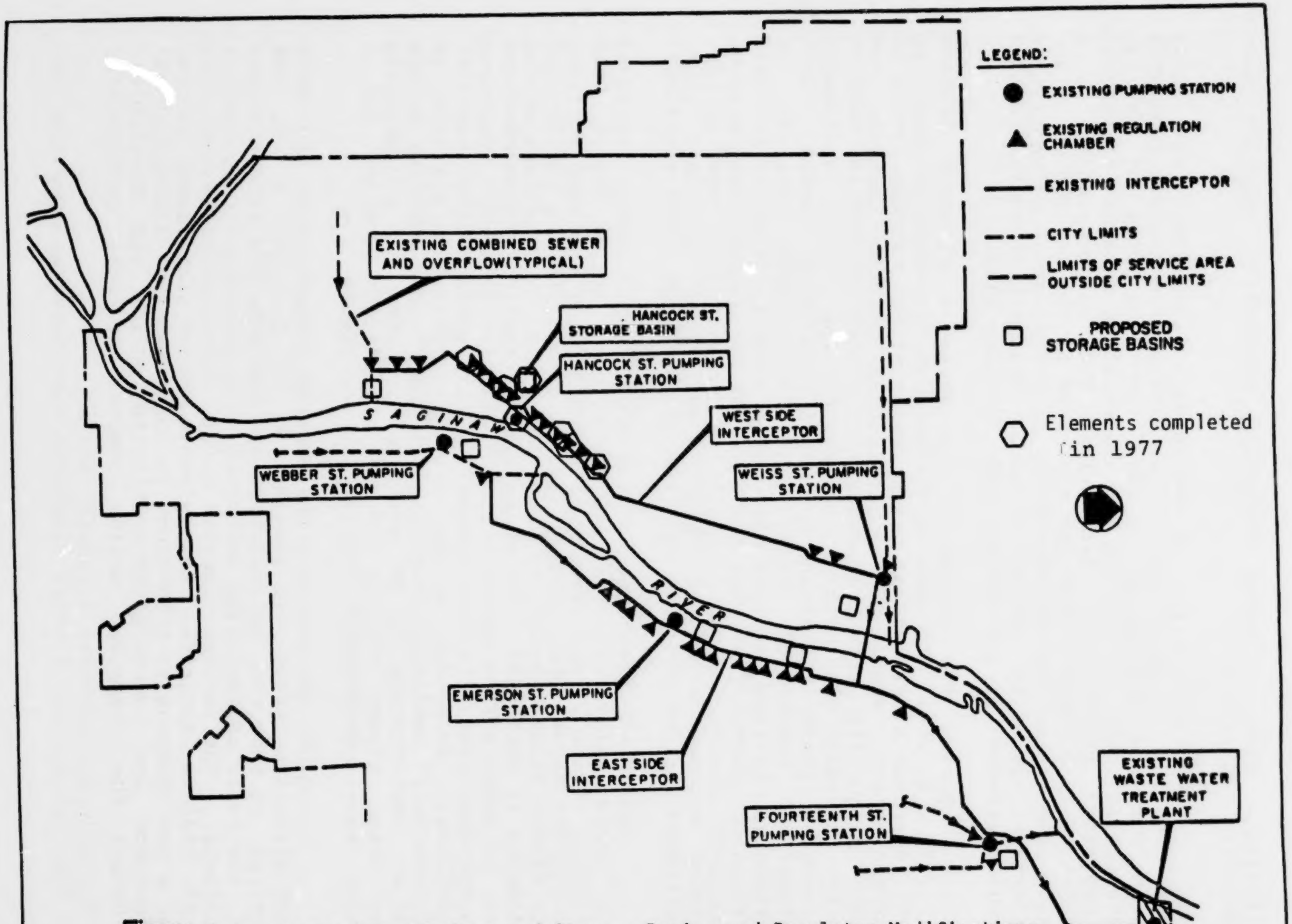
Figure 1.1 Location of Saginaw, Michigan.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● EXISTING PUMPING STATION
▲ EXISTING REGULATION CHAMBER
EXISTING INTERCEPTOR
CITY LIMITS
LIMITS OF SERVICE AREA OUTSIDE CITY LIMITS
□ PROPOSED STORAGE BASINS
⬡ Elements completed in 1977
EXISTING COMBINED SEWER AND OVERFLOW(TYPICAL)
HANCOCK ST. STORAGE BASIN
HANCOCK ST. PUMPING STATION
WEST SIDE INTERCEPTOR
WEISS ST. PUMPING STATION
SAGINAW
RIVER
WEBBER ST. PUMPING STATION
EMERSON ST. PUMPING STATION
EAST SIDE INTERCEPTOR
FOURTEENTH ST. PUMPING STATION
EXISTING WASTE WATER TREATMENT PLANT

Figure 1.2 Location of Proposed Storage Basins and Regulator Modifications

facility, other proposed detention facilities and the five modified regulators.

The objectives of the Phase 1 (1979-1981) EDP study were twofold: a) preparation of an alternative overall framework plan for the management of CSO from the City of Saginaw considering, but not limited to, the existing concepts of storage/ treatment/ chlorination facilities; and b) preparation of a detailed preliminary plan for a portion of the envisioned city-wide plan, entailing implementation of a full-scale demonstration treatment facility devised in such a way that it would form a logical element of the overall CSO Facility Plan.

During the course of the Phase 1 planning efforts (1,2,3) by EDP, the most cost-effective City-wide programs of CSO abatement alternatives, were developed for varying levels of system costs. Runoff simulation studies indicated that the two most singularly effective measures for handling CSO in Saginaw were increased wet weather loadings to the City's wastewater treatment plant (WWTP) and inclusion of actions termed, "Best Management Practices" (BMP). The BMP program consists of extensive in-line storage and system modifications to increase hydraulic capacity so that the WWTP can handle more wet weather combined sewage. Roughly 2 million cubic feet of new in-line system storage would be generated as a result of this plan. Estimated cost of BMP plan equalled $2.6 million (1984 ENR=4300).

After implementation of the complete BMP plan, residual CSO[*] would still remain at six locations: Weiss Street Pump Station, Weiss Street gravity line, Emerson Street Pump Station, Fourteenth Street Pump Station,[**] Webber Street Pumping Station and the Fraser Street Regulator. See Figure 1.3 for locations of the elements of this plan. The Phase 1 investigation recommended the use of swirl solids concentrating facilities at these locations as the most cost-effective technology for abating these residual overflows. Suspended solids removal optimization analysis of various swirl configurations/sizes at the various overflow points (after implementing the BMP program) resulted in pin-pointing the Weiss Street area as the most significant location for a major swirl concentrator facility. See Figure 1.4.

Although the proposed BMP program elements were the most cost-effective means for abating CSO in Saginaw, U. S. EPA desired at that time to implement and evaluate the effectiveness of the swirl concentrator technology for reducing CSO pollutant loadings. The 24-foot diameter swirl concentrator (design flow - 40 cfs) in Lancaster, Pennsylvania showed significant promise as a new inexpensive technology in comparison to sedimentation/ detention for reducing CSO settleable solids and floatables. Although the suspended solids removal results, particularly

[*] For moderate to heavy intensity storms.

[**] Pump station and gravity discharge considered as composite CSO.

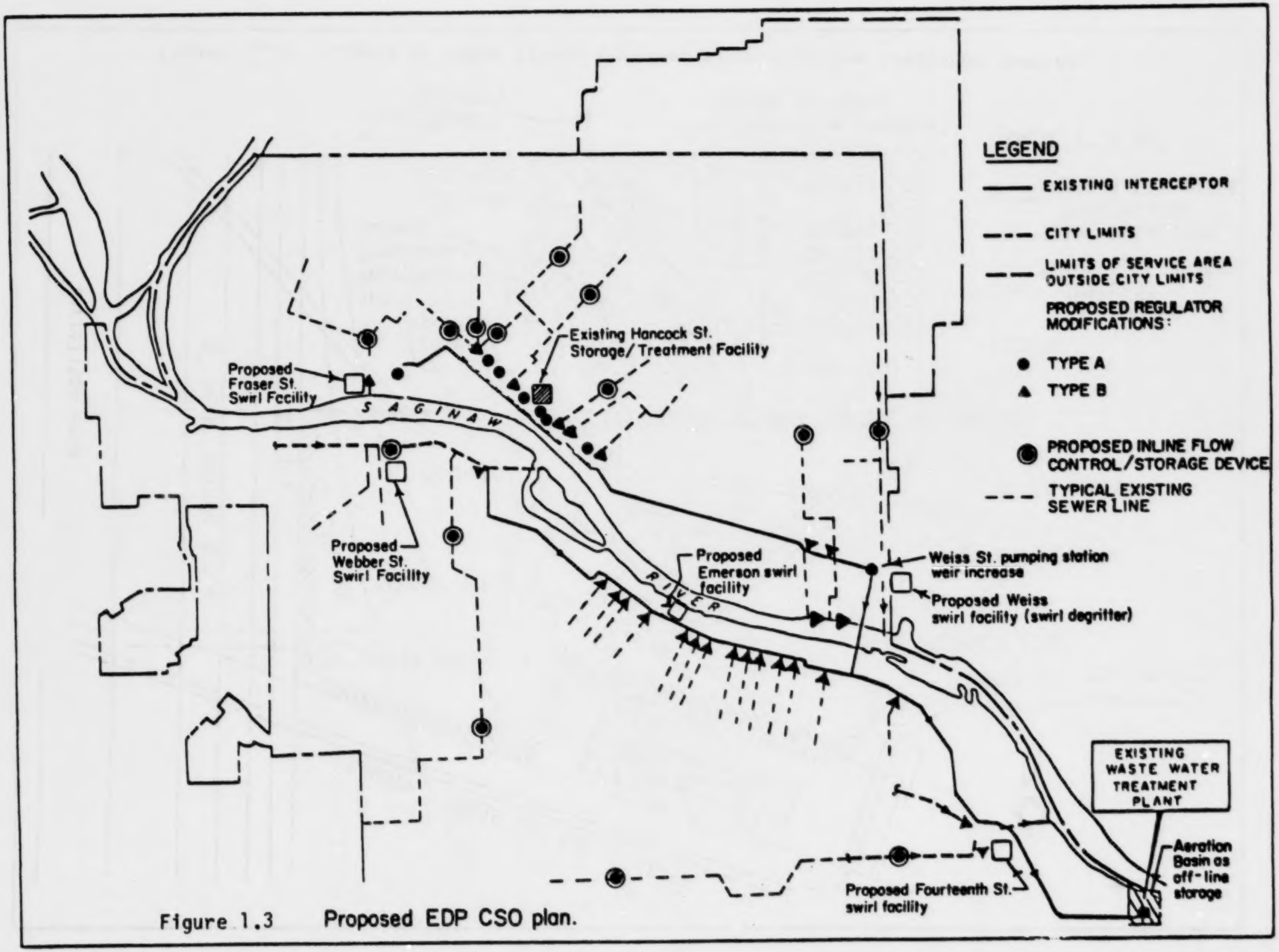

LEGEND
EXISTING INTERCEPTOR
CITY LIMITS
LIMITS OF SERVICE AREA OUTSIDE CITY LIMITS
PROPOSED REGULATOR MODIFICATIONS:
● TYPE A
▲ TYPE B
PROPOSED INLINE FLOW CONTROL/STORAGE DEVICE
TYPICAL EXISTING SEWER LINE
Existing Hancock St. Storage/Treatment Facility
Proposed Fraser St. Swirl Facility
S A G I N A W
R I V E R
Proposed Webber St. Swirl Facility
Proposed Emerson swirl facility
Weiss St. pumping station weir increase
Proposed Weiss swirl facility (swirl degritter)
EXISTING WASTE WATER TREATMENT PLANT
Aeration Basin as off-line storage
Proposed Fourteenth St. swirl facility

Figure 1.3 Proposed EDP CSO plan.

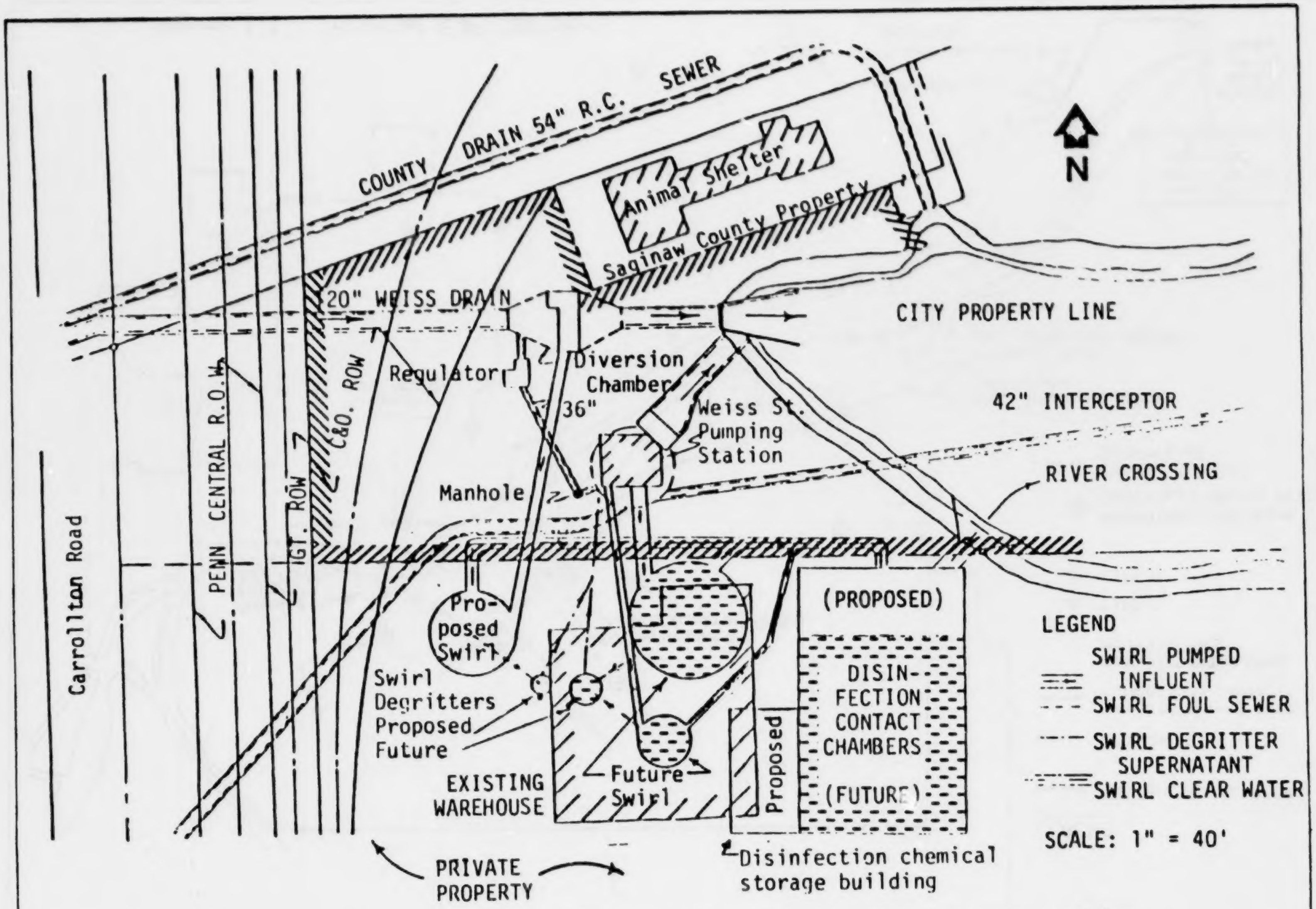

N
COUNTY DRAIN 54" R.C. SEWER
Animal Shelter
Saginaw County Property
20" WEISS DRAIN
CITY PROPERTY LINE
Regulator
Diversion Chamber
36"
Weiss St. Pumping Station
42" INTERCEPTOR
RIVER CROSSING
Manhole
Carrollton Road
PENN CENTRAL R.O.W.
GT. ROW
C&O. ROW
Proposed Swirl
Swirl Degritters Proposed Future
EXISTING WAREHOUSE
Future Swirl
Proposed
(PROPOSED)
DISIN-FECTION CONTACT CHAMBERS
(FUTURE)
Disinfection chemical storage building
PRIVATE PROPERTY
LEGEND
SWIRL PUMPED INFLUENT
SWIRL FOUL SEWER
SWIRL DEGRITTER SUPERNATANT
SWIRL CLEAR WATER
SCALE: 1" = 40'

Figure 1.4 Layout of Weiss Street combined sewer overflow treatment complex.

during "first flush" conditions, were extremely favorable at the swirl concentrator facility in Lancaster, the evaluation program was plagued with numerous flow measurement instrumentation problems. Since a number of swirl concentrators were being incorporated into designs throughout the U. S. at that time, U.S. EPA believed that the proposed evaluation program at Saginaw would answer any unresolved questions remaining from the Lancaster experience. Moreover, the Canadian Ministry of the Environment at the time was seriously considering this technology for widespread utilization in the Quebec Province (one facility was constructed in Quebec City in 1984 and more facilities (1985) are in design). A comprehensive and complete portrayal of swirl performance was desired for technology transfer. It was also desired by U.S. EPA at that time to construct and evaluate a swirl concentrator of larger physical dimensions than the device at Lancaster to expand the evaluation experience for this emerging technology.

The Phase 1 study (1) also recommended within the budgetary limitations (of the demonstration funding) three alternative swirl concentrator demonstration facilities located at the Weiss Street gravity overflow, at the Weiss Street Pump Station discharge and at the gravity discharge at Fourteenth Street. Estimated Construction costs (1984) including electrical service contingency and excluding disinfection chemicals for the three facilities equalled \$3.1 ,\$2.6, and \$2.38 million(ENR=4500) , respectively. No land costs were included. The facility at the Weiss Street gravity overflow was preferred.

Because of the significant cost savings for City-wide CSO control program reported from the Phase 1 study and as a mechanism to facilitate Section 201 Construction Grants funding to partially support the costs of the proposed demonstration project at Weiss Street, EDP was instructed by the City of Saginaw and U.S. EPA in mid 1980 to expand the Phase 1 study into a proposed CSO Facility Plan for the City of Saginaw (2). In January, 1981 the proposed EDP CSO Facility Plan was adopted by the City of Saginaw.

Although the magnitude of the overflow at Weiss Street greatly exceeded those at other locations, City-owned land space to construct the proposed facility was extremely limited. Design of the proposed swirl concentrator facility at Weiss Street was curtailed due to the envisioned long and protracted period required to acquire the necessary land.

In mid 1981 EDP was instructed by the City and U.S. EPA to re-direct efforts for placement of the proposed swirl concentrator facility at an alternative location within the City. The alternative facility was nevertheless to be designed and constructed in such a manner as to be a logical and congruent element of the overall adopted CSO Facility Plan. The Webber Street Pumping Station area was chosen as the new location to place the proposed demonstration facility. An analysis of this catchment was conducted supported by detailed field measurement programs (3). In 1981-1982 EDP prepared construction plans, specifications and bid documents for a new treatment complex consisting of an in-line diversion structure controlled by vortex flow regulation devices to direct flow to the existing pump station, a new 80-cfs low lift pumping plant, a 38 foot diameter swirl concentrator with stainless steel inner assemblies and ancillary piping and connections. See Figure 1.5. The cost of the facility was estimated to be approximately $2.0 million (ENR=4500 , 1985)

Approximately $.75 million was remaining from the 108 grant and additional funding was requested from the 201 Construction Grants Program. Eligibility status for the proposed project was poor and it was forcasted that the low status would remain for the next several years. It was finally decided by U.S. EPA that the proposed demonstration swirl concentrator project be indefinitely postponed.

In early 1983, U.S. EPA and the International Joint Commission (IJC) (U.S. and Canada) became interested in developing phosphorous limitations for the Great Lakes revolving around a number of promising BMP practices controlling agricultural and urban runoff phosphorous loadings. Sewerage system management employing system controls to inexpensively maximize transient in-line storage with bleedback to WWTP's with high performance treatment was identified and viewed by U.S. EPA as a promising management prospect in view of the continuing low federal funding profile for the next several years. Since a number of the major sewerage systems impacting the Great Lakes are combined, relatively flat and discharge to high performance WWTPs, the notion of evaluating inexpensive system storage and hydraulic optimization in Saginaw was attractive, feasible and logically extendable to other areas within the Great Lakes region, if proven to work.

On instruction from U.S. EPA and the City in April, 1983, EDP embarked on Phase 2 of the overall program and prepared a preliminary concept plan to best spend the remaining grant monies to implement and evaluate a significant portion of the BMP plan. The aim was to accomplish as much as possible with remaining monies. Runoff simulation studies were performed to evaluate numerous schemes varying the proposed modified control points. Optimization analysis identified a plan for final selection which included sewerage system modifications on only one side of the Saginaw River (4). In May, 1983 EDP met with the City and U.S. EPA to discuss the envisioned plan which would

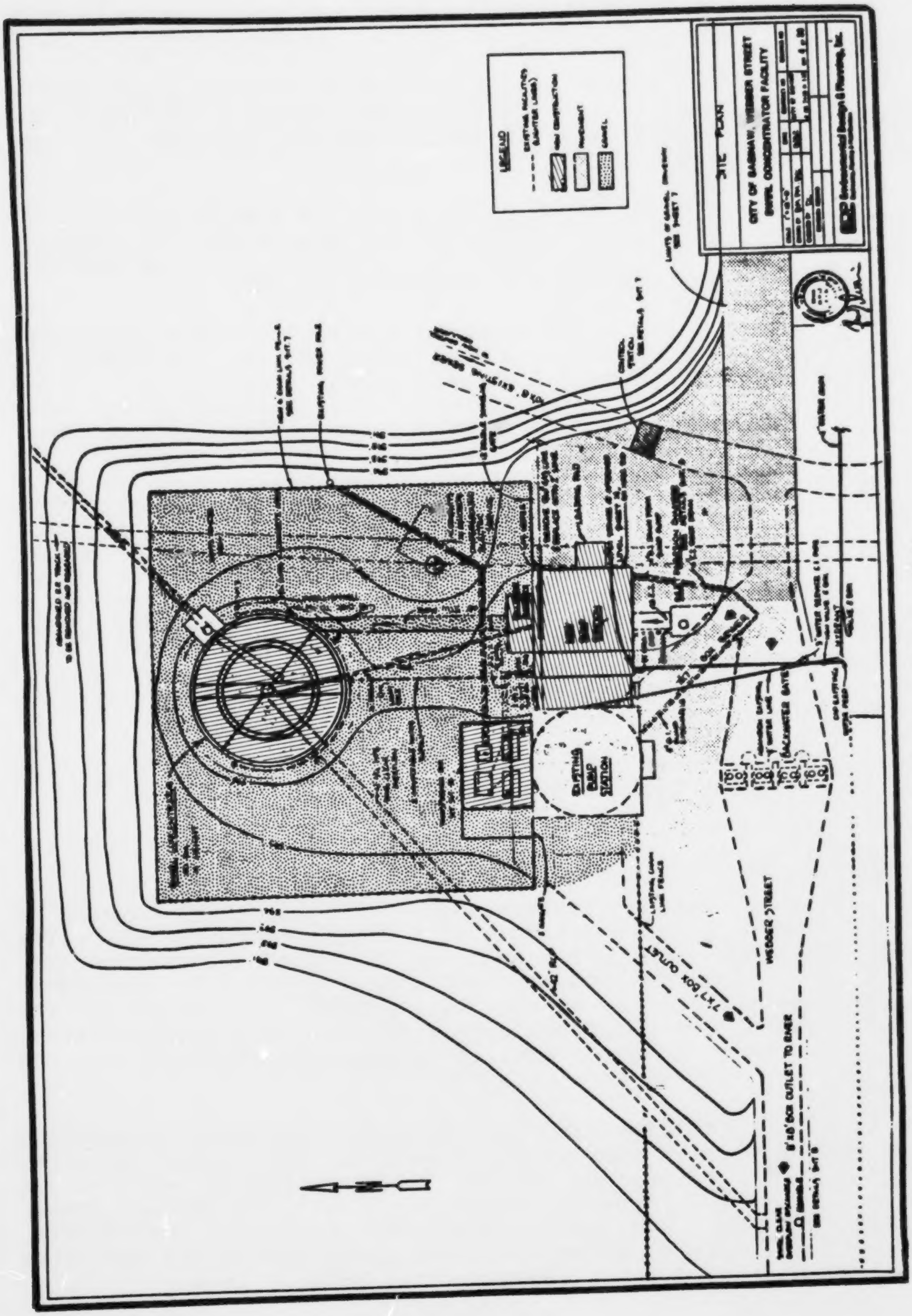

SITE PLAN
CITY OF SAGINAW, WEBBER STREET
SWIRL CONCENTRATOR FACILITY
Environmental Design & Planning, Inc.
LEGEND
EXISTING PUMP STATION
WEBBER STREET
7'x7' BOX OUTLET
8'x8' BOX OUTLET TO RIVER
BACKWATER GATE

Figure 1.5 Webber Street Swirl Facility

realize about half of the pollution control effectiveness of the total BMP plan. As a result EDP was instructed by the City and U.S. EPA to proceed with an implementation program for the "reduced BMP" Plan.

EDP prepared construction plans and specifications. In March 1984, the construction contract was awarded to Gary D. Steadman Inc. of Bay City, Michigan. The construction began in April, 1984 and was completed in September of 1984.

During Phase 3 EDP directed a detailed pre-construction and a post-construction field evaluation program performed by City of Saginaw personnel. Rain event statistics, river elevations, sewer depths, regulator chamber hydraulics and water quality parameters were recorded at numerous locations throughout the City. Pre- construction data was used to calibrate the simulation model for discrete events. The model was then run for a full year simulation to estimate base pre-construction sewerage system overflows during storm conditions. Post-construction data was evaluated and compared to predicted overflow quantity and quality values based on the modified model reflecting the pre-construction measurements. Model simulation data and field data were comparable such that further model calibration was not required. Full year model simulations were then performed to indicate the effectiveness of the implemented reduced BMP plan followed by simulations predicting the effectiveness of completing the entire BMP plan.

This report documents the scope of work and the simulation results for the five year, three Phase project.

1.3 Details of Phase 1 Evaluation

The purpose of the Phase 1 evaluation was to develop a cost effective plan of action that mitigates impacts on water quality caused by CSO. The Facility Planning Area encompasses the City of Saginaw land mass including pertinent contingent township areas tributary to the Saginaw WWTP. The scope of the plan was limited to overflows within the facility planning area, as well as to the water quality parameters defined by the objectives and issues.

The CSO Facility Plan was developed using the parameters of dissolved oxygen and oxygen demanding materials, coliform bacteria, and suspended solids and settleable solids as indicators of CSO impacts and target levels to be attained. Although CSO has many other associated pollutant parameters, such as heavy metals, chlorinated organics, pesticides and other toxics, this study did not qualitatively address them.

The scope of the Phase 1 report was limited to an assessment and evaluation of water quality impacts resulting from CSO and then the development of a plan to mitigate those impacts caused by CSO. It was recognized that water quality is impacted by upstream conditions from areas outside the combined sewer study area, point source emissions and direct stormwater runoff. The study made an assessment of the relative impact from these areas relative to CSO but did not pursue them in the development of the Recommended Plan.

1.3.1 Project Approach

Phase 1 of the project was developed in the following manner:

1. Evaluation of existing conditions
2. Simulation modeling of CSO and water quality impacts
3. Evaluation/optimization of abatement alternatives
4. Development of the recommended plan

Existing facilities were evaluated to generate an accurate description of the sewerage system including physical and operating characteristics of the WWTP, the Hancock Street CSO storage/treatment facility, all pump stations and all regulator chambers.

Formulation of the basis of design for CSO overflow control facilities required input of data, with regard to the quality and quantity of dry weather flows, so that network models could be calibrated. Efforts were made to determine the quality of dry weather flows at various points in the City of Saginaw system and to isolate catchment areas including the Saginaw WWTP. A field measurement program was performed to enable estimates of dry weather flows per catchment area.

Analyses were performed to assemble the wet weather predictive design features including rainfall characterization, determination of runoff parameters for the catchment areas, and evaluation of regulator chambers and pumping stations.

A comprehensive mathematical computer simulation program was developed to help determine overflow discharges and pollutant concentrations as a result of hypothetical occurrence of various size design storms. The model provided an accurate representation of the physical sewerage system, but also provided an opportunity to determine the benefit of proposed pollution abatement alternatives. The value of modeling lies in the ability to predict the outcome of dependent variables as a result of hypothetical modification of the system. These projections allowed for more efficient evaluation of abatement alternatives formulated to meet established goals, concepts and criteria. Through this process the best alternatives were isolated for further cost-benefit analysis.

The Saginaw Stormwater Management Model, developed by EDP, provided an inexpensive and flexible tool for preliminary planning and sizing of stormwater facilities. The model provided a means by which the stormwater problem could be analyzed and introduced time and probability to the quantity, frequency and duration of overflow events by using actual rainfall data collected over a long period of time. It was used to screen alternatives based on varying storm conditions.

Feasible abatement programs for CSO pollution control required evaluation to select the most cost-effective configuration and scale to achieve water quality goals. This study examined the effectiveness and costs of selected alternatives, and combined the estimates to develop a recommended facility CSO pollutional abatement program.

The major components of the Phase 1 recommended CSO abatement program are as follows:

- BMP program implementation
- Swirl Concentrator Facilities at:

	Design Capacity
Weiss Street Pump Station	300 cfs
Weiss Street Gravity Line	100 cfs
Emerson Street Pump Station	300 cfs
Fourteenth Street Pump Station *	50 cfs
Webber Street Pump Station	50 cfs
Fraser Street	50 cfs

*** Pump Discharge and Gravity Line CSO handled by single facility.**

1.3.2 Description of Complete BMP Program

The BMP program as proposed by EDP and adopted as part of the Facility Plan, incorporates low-level structural improvements and management practices that can be implemented at relatively low cost. The concept of the program is to maximize the use of existing WWTP facilities before further CSO treatment facilities are implemented. The program maximizes in-line storage already available in the system conduits and maximizes the conveyance of storm flows to the WWTP. To actualize this plan, the existing regulators were to be modified, new flow control devices were to be installed to increase in-line storage, weir modifications were to be accomplished at the Weiss Street Pump Station to increase flow across the river, and the operating procedure of utilizing the extra aeration basins at the WWTP for off-line storage implemented. (The City was utilizing this option at the time it was identified by EDP.)

It was envisioned to alter 24 regulators (See Figure 1.3) to yield a constant discharge* during initial storage filling, with excess flows beyond available storage** diverted to the interceptor rather than the river. In addition, the five modified Hancock area regulators (modified as part of the Hancock Street Detention Facility Implementation in 1977) were to be replaced with constant underflow discharge devices (vortex valve flow regulators) to increase the "first flush" diversion to the interceptor and maximize the use of the interceptor throughout the storm event. Furthermore, six other regulators of smaller size were to be altered with weirs to divert most flows to the interceptor. Fourteen upstream structures with vortex valve flow regulators were to be constructed to yield additional in-line flow control for storage upstream of the regulator chambers. The resulting changes to the regulator operations would achieve the maximum useable in-line storage potential of the system.

Except in extreme flow conditions, the only possible points of overflow as a result of these potential actions are at Fraser, Hancock, Weiss (pump and gravity), Webber, Emerson and Fourteenth Streets. Weir elevations at Webber and Fourteenth Street overflow points were to be raised slightly to insure that the high post-rain discharges from upstream in-line storage devices are totally diverted to the interceptor. The regulator chamber at Fraser Street will be unaltered although the regulating device was to be replaced and a flow control chamber constructed only a short distance upstream. Figure 1.6 and 1.7 detail typical regulator modifications and the original underflow control device to be removed.

An increased weir elevation of five feet was to be imposed at the Weiss Street Pumping Station to reduce pumped overflows at that point and increase the flow across the river crossing and into the East Side Interceptor. The aeration basins at the WWTP were to continue to be utilized as additional off-line storage devices. The lift pumps at the WWTP were to pump from the interceptor up to maximum hydraulic capacity until the aeration basins are filled. The aeration tank contents are returned to the primary treatment tanks at the end of the event.

* Using vortex flow control devices.
**For small to moderate storm events.

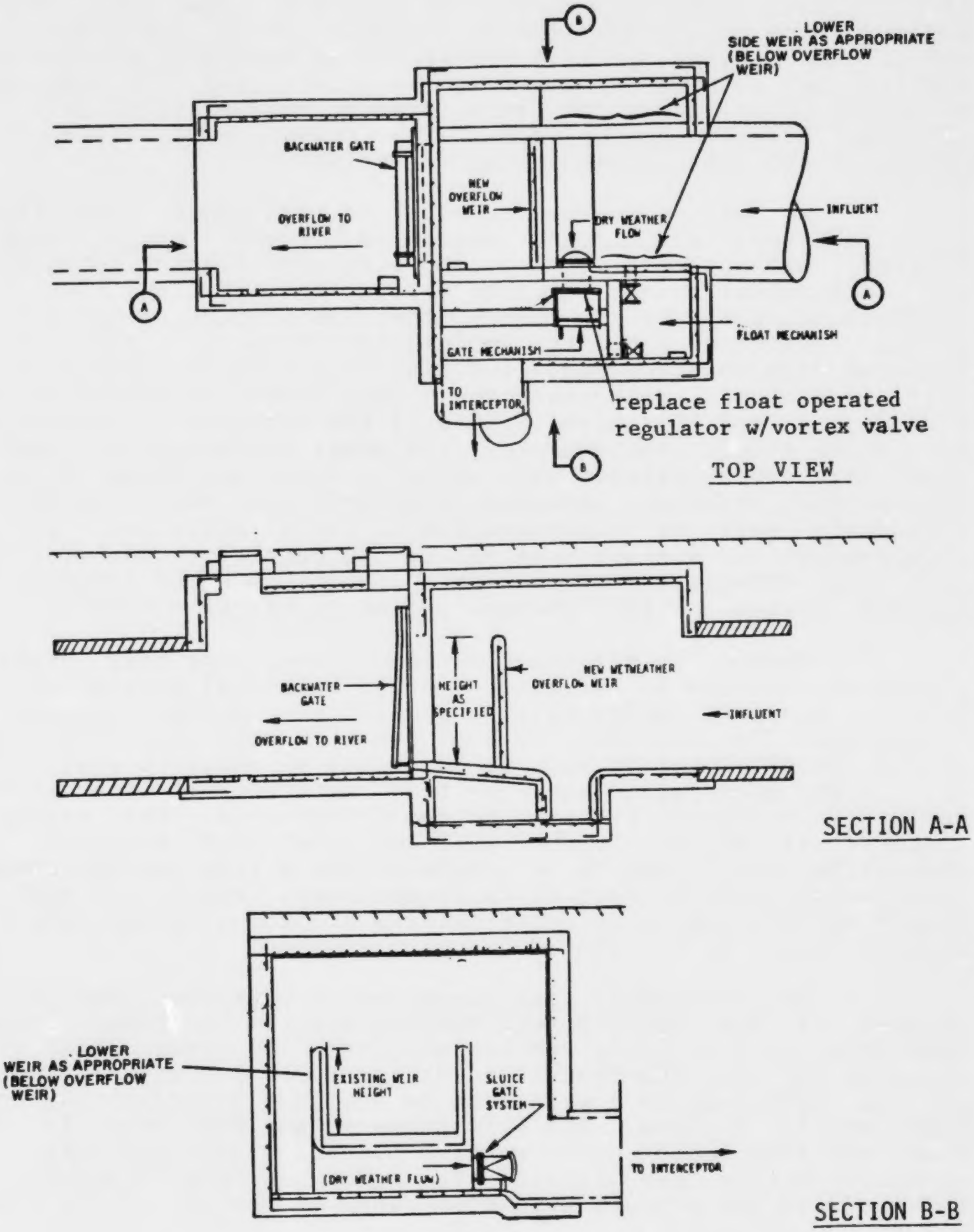

LOWER
SIDE WEIR AS APPROPRIATE
(BELOW OVERFLOW
WEIR)
BACKWATER GATE
NEW OVERFLOW WEIR
OVERFLOW TO RIVER
DRY WEATHER FLOW
INFLUENT
FLOAT MECHANISM
GATE MECHANISM
TO INTERCEPTOR
A
B
replace float operated
regulator w/vortex valve
TOP VIEW
BACKWATER GATE
OVERFLOW TO RIVER
HEIGHT AS SPECIFIED
NEW WETWEATHER OVERFLOW WEIR
INFLUENT
SECTION A-A
LOWER
WEIR AS APPROPRIATE
(BELOW OVERFLOW
WEIR)
EXISTING WEIR HEIGHT
SLUICE GATE SYSTEM
(DRY WEATHER FLOW)
TO INTERCEPTOR
SECTION B-B

Figure 1.6 Typical in-line storage device (regulator).

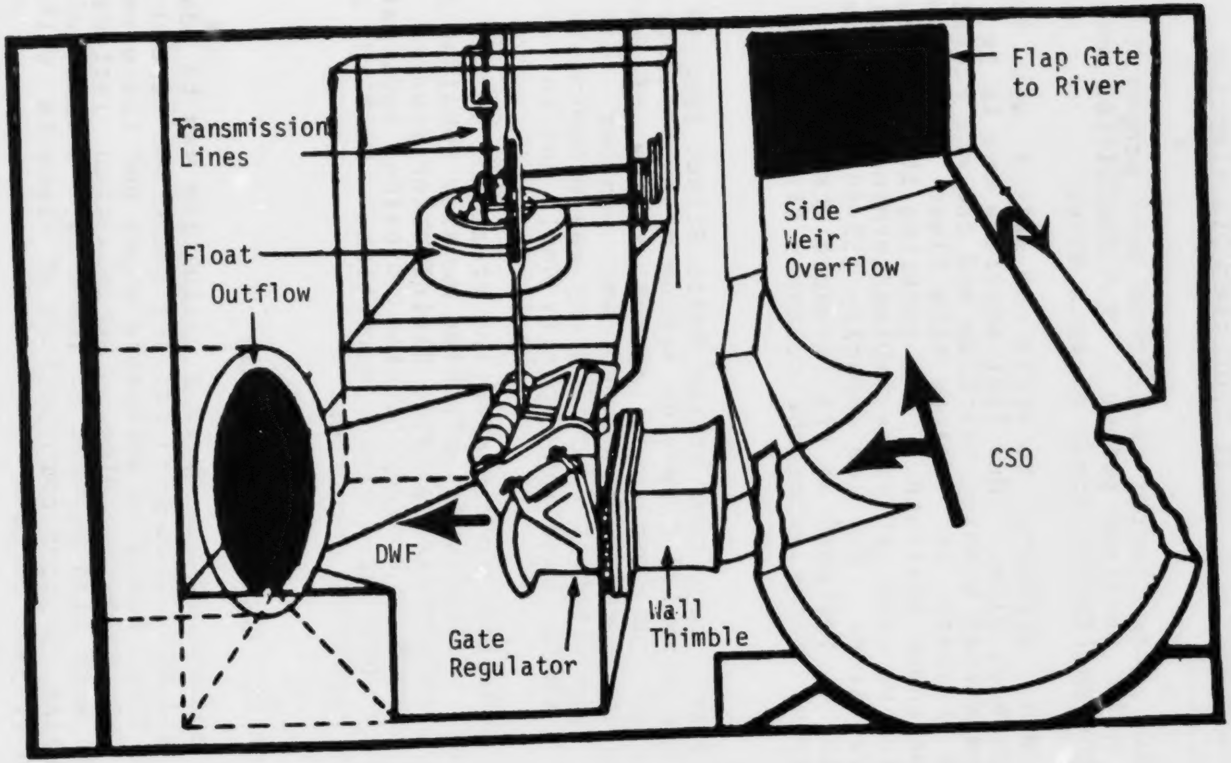

Transmission Lines
Float
Outflow
DWF
Gate Regulator
Wall Thimble
Flap Gate to River
Side Weir Overflow
CSO

Figure 1.7 Typical Float Actuated Type Flow Regulator

1.3.3 Swirl Concentrator/Swirl Degritter/Disinfection Facilities

Swirl concentrator facilities were proposed at each of the six major overflow points. The facilities were to be tailored to the particular needs of each site.

Pumping stations to drive the swirls were considered necesssary at each site. New lift stations were to be included for the Weiss Street gravity overflow and for the Fraser Street overflow. A battery of various size fixed speed pumps were considered for these stations. The existing 100-cfs fixed speed pumps were to be used (after revamping/turning) for all other stations as required. New low capacity (25-40 cfs) fixed speed pumps were to be included in all revamped stations to provide variable pumping range using the revamped fixed speed 100-cfs pumps.

A swirl degritter at the Weiss Street complex may be necessary if the proposed City-wide system of swirl concentrators were implemented and pending a careful review of WWTP grit removal capacity under sustained wet weather high flow conditions. This degritter contingency was proposed to ensure that the WWTP's solids handling capacity would not be overtaxed.

In addition to disinfection treatment at the Hancock Street Facility, overflow disinfection was also considered at the six aforementioned overflow points. Disinfection process is to be sequential - staged chlorine/chlorine dioxide application (8 ppm/2 ppm) with total of 2 minute detention. Baffled chambers are to provide necessary detention.

1.4 Details of the Phase 2 Evaluation

The purpose of the Phase 2 evaluation was to determine the optimum implementation plan for the reduction of CSO impacts to the Saginaw River based on available money and the elements of the overall CSO Facility Plan. Optimization included cost estimation of each plan element followed by a series of model simulations using various combinations of elements within the total allotted project budget.

As a first step, additional field evaluations of flow patterns at various regulator chambers were performed allowing further calibration of the simulation model. A limited field investigation was performed including continuous water elevation recording at several regulators on each side of the Saginaw River. This data was combined with information routinely collected by City personnel at the Weiss Street and Hancock Street Pump Stations and at the WWTP to estimate area wide hydraulic conditions. Simulations then indicated the optimum configuration for a reduced BMP implementation plan. This plan consisted of modifications to the Saginaw sewerage system only on the West Side of the Saginaw River. See Figure 1.8 for locations controlled as part of the reduced BMP plan.

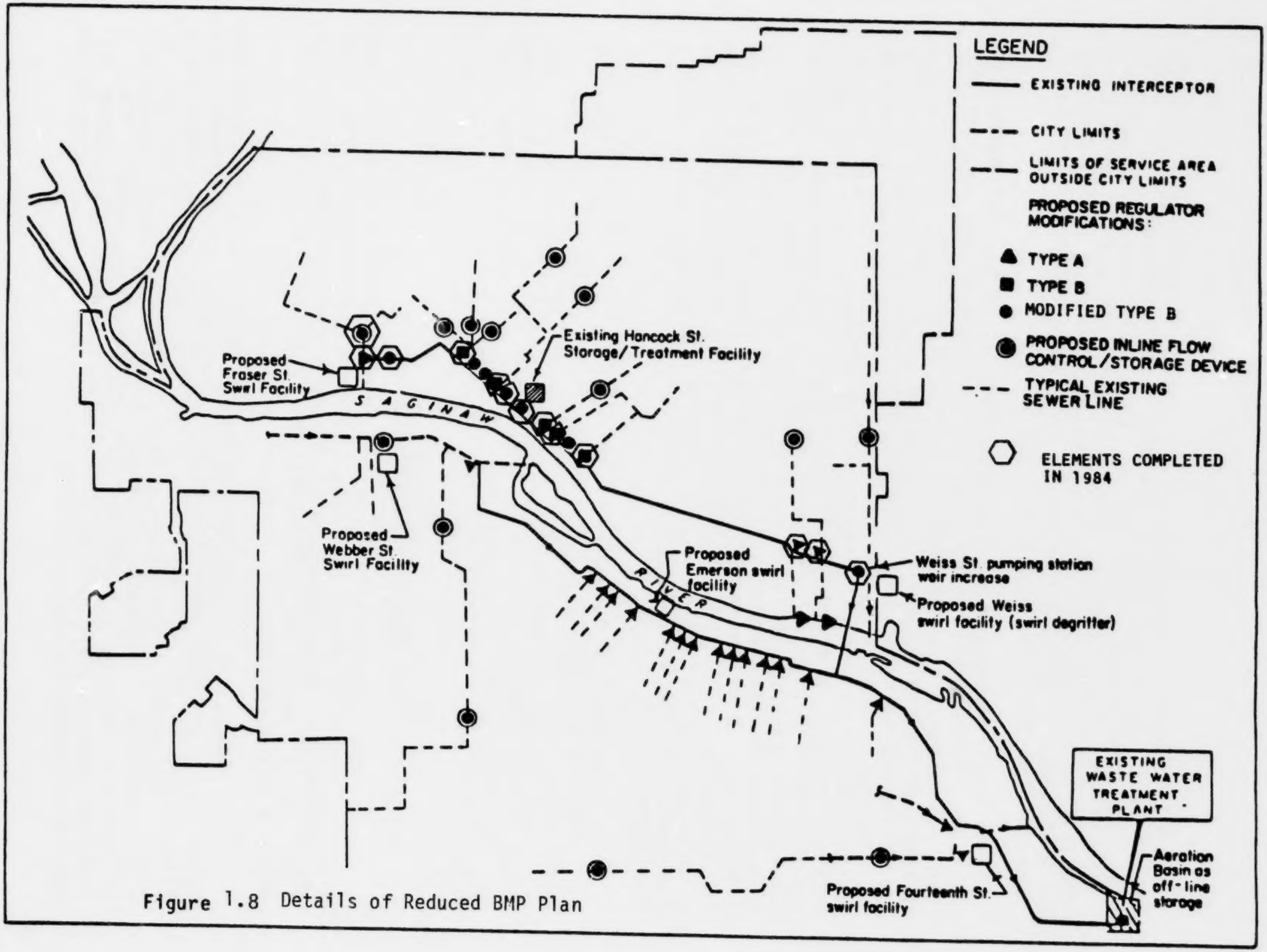
Since the entire BMP plan was not to be implemented at this time, it was important that all elements of the reduced BMP plan be designed and implemented such that no modifications will be required in the future. Furthermore, the concept of the total BMP plan was to control the system such that overflows (except for extreme events) would occur only at major pumped discharge points. Since it was not envisioned to modify the Emerson Street Pumping Station at this time (for automatic "turn-on"), the reduced BMP plan was designed such that no additional capital expenditure for pumpage and/or operational problems would occur as a result of the implementation. In short, all elements of the reduced BMP plan were to be permanent and not subject to modifications. The entire BMP plan may be accomplished in the future and all elements of the reduced BMP project were to be designed so that the conveyance/pumpage characteristics remain unchanged.

As part of the reduced BMP plan, 12 of 35, of the City of Saginaw's combined sewer regulators were modified in accordance with the CSO Facility Plan Concept. One of the 14 upstream in-system type structures was also constructed. The Weiss Street Pumping Station weir/wet well operation was modified to increase river crossing discharge so that more of the West Side "first flush" is transmitted to the WWTP.

1.5 Details of the Phase 3 Evaluation

The Phase 1 EDP sewerage system simulation model was used to estimate City-wide emissions pre/post reduced BMP plan implementation. Flow measurements using automatic equipment were made at a number of representative regulators as part of the Phase 2 measurements.

After the reduced BMP plan project was implemented, Phase 3 flow measurements were conducted to document the effectiveness of the implemented controls on a representative basis. Automatic continuous level sensors were installed in representative regulator chambers to indicate hydraulic conditions during dry weather and wet weather conditions. A limited post-implementation water quality sampling program was conducted using automatic sampler techniques to obtain composited


LEGEND
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PROPOSED REGULATOR MODIFICATIONS:
▲ TYPE A
■ TYPE B
● MODIFIED TYPE B
◉ PROPOSED INLINE FLOW CONTROL/STORAGE DEVICE
TYPICAL EXISTING SEWER LINE
⬡ ELEMENTS COMPLETED IN 1984
Proposed Fraser St. Swirl Facility
SAGINAW
RIVER
Existing Hancock St. Storage/Treatment Facility
Proposed Webber St. Swirl Facility
Proposed Emerson swirl facility
Weiss St. pumping station weir increase
Proposed Weiss swirl facility (swirl degritter)
EXISTING WASTE WATER TREATMENT PLANT
Aeration Basin as off-line storage
Proposed Fourteenth St. swirl facility

Figure 1.8 Details of Reduced BMP Plan

samples from one major overflow point (Weiss Street Pumping Station), at one regulator (Adams Street) and at the WWTP. Samples were analyzed for TSS, BOD and total/soluble phosphorus by the City of Saginaw.

These documentary measurements were used to adjust the simulation model as necessary to note the technical effectiveness of the flow controllers/in-line storage. The simulation model was then re-applied to the entire system to estimate residual emissions as a result of the plan. Thus, Phase 2 pre-implementation flow measurements were used to sharpen the planning/design phase and to provide a "pre-project" documentation. Phase 3 post-implementation measurements were again used for documentation purposes as well as to "fine-tune" the simulation model to incorporate the actual effectiveness of the plan elements. The resulting simulation model thus estimates residual CSO emissions incorporating to a reasonable extent, actual configuration effectiveness.

Detailed observations were also noted by City of Saginaw maintenance personnel during routine post storm inspection of regulator chambers to determine physical conditions. One aspect of the BMP plan was to include low maintenance flow regulating devices to replace the existing float operated type. These existing devices require frequent maintenance and are subject to failure. The new vortex valve flow regulating devices performed satisfactorily and have reduced maintenance problems.

Structural changes to regulator chambers were intended to improve system storage capacities and flow hydraulics without causing excessive surcharge or in-system sedimentation. Degree of backwatering was noted at numerous locations upstream in the sewerage system. Debris accumulations in all regulator areas and sedimentation in dry weather flow channels were particularly noted. These observations served to document the reduction in maintenance labor provided by reduced BMP plan implementation.

The post implementation documentary effectiveness of the reduced BMP plan improvements in terms of increased wet weather pollutant loadings to the WWTP was close to pre implementation predictions. In sum, expenditures of less than $516,000 under the reduced BMP plan for low level sewerage system modifications (entailing a significant portion of proposed controls for one-half of the City) resulted in decreasing the system-wide total phosphorous wet weather loadings to the river by about 10%. No adverse conditions resulted from the plan. The concept of inexpensive system controls to maximize transient in-line storage of combined sewage with bleedback to a high performance WWTP worked well and is recommended as a management practice.

1.6 Reader's Guide to Report

Report conclusions and recommendations are presented in Chapters 2 and 3, respectively. A general description of the Saginaw sewerage system is presented in Chapter 4. A review of the combined sewer overflow control technologies considered as part of the Facility Planning effort is presented in Chapter 5. Details of the modeling approach used throughout the project are presented in Chapter 6. Design and implementation construction details for the reduced BMP plan are presented in Chapters 7 and 8, respectively. Details of the post construction measurement evaluation program are presented in Chapter 9. WWTP phosphorous removal performance determinations conducted during this period are reported in Chapter 8. Final modeling results documenting overall pollutant removal efficiency for the implementated plan are given in Chapter 10. Details of stormwater management investigations intended to alleviate basement flooding problems due to surcharging sewers in upstream areas which also effect the magnitude of downstream CSO levels are reported in Chapter 11.

CHAPTER 2

CONCLUSIONS

EDP has completed a five year 3 Phased study of CSO pollution from the City of Saginaw, Michigan and concludes that CSO pollution in Saginaw is controllable at reasonable costs and that water quality improvements in the Saginaw River and public health gains will be significant. The conclusions are based on the analyses performed over the past five years, actual data collected concerning CSO and river quality, previous analyses of the water quality of the region and partial implementation and evaluation of proposed system modifications. It should be noted that the conclusions stated below include those developed during earlier EDP investigations leading to the development and adoption of a CSO Facility Plan for Saginaw. These earlier conclusions are presented to depict a perspective framework for the more relevent and important conclusions resulting from the implementation/evaluation of the reduced BMP program which is an integral element of the overall CSO Facility Plan.

Conclusions are detailed in the following order:

PLANNING & DESIGN

A. Combined Sewer System Data Reduction Analysis
B. CSO Management Model

IMPLEMENTATION & EVALUATION

C. Implementation of Reduced BMP Plan
D. Evaluation of Implemented BMP Plan

FUTURE PROSPECTUS

E. Optimization and the Recommended Facility Plan
F. Cost Analysis of Completed and Proposed Future Work
G. Effects of Facility Plan Implementation on River Quality

A. Combined Sewer System Data Reduction Analysis

(1) Extensive data concerning the Saginaw combined sewer system, intercepting sewer systems, and storm and sanitary waste water treatment facilities were collected, reduced and analyzed. Available data were supplemented by several EDP measurement and inspection programs.

Wastewater Treatment Plant

(2) The secondary WWTP had the capacity to handle combined sewage volumes greater than generally flowing to the plant. An analysis of operating records indicated that both hydraulic and treatment capacity for additional loadings were available without impairment of overall WWTP treatment efficiency. Increasing the flow of combined sewage to the WWTP was a primary goal of this study. Pollutant loads to the river are expected to be significantly reduced by increasing flow to the WWTP and storing excesses within the system until throughput capacity becomes available.

(3) Review of daily operating records (long term) indicated that the BOD removal for the WWTP averages approximately 89% for both wet days and dry days. It was concluded that the WWTP could handle higher wet weather flows without compromising the plant´s pollutant removal capabilities.

(4) Review of WWTP data indicate wet weather total phosphorous removal of 50% without AWT . Non- soluble phosphorous removals are 95% but no soluble phosphorous removal is achieved. The non soluble fraction averages 54% of the total phosphorous.

(5) Advanced Wastewater Treatment (AWT) (pickling liquor or FeSO4 addition) is used, as required at the WWTP to meet effluent standards. Phosphorous removals average 75% with AWT and the entire additional level removed beyond secondary treatment rates is attributable to a 50% reduction of soluble phosphorous.

(6) A decrease in non-soluble phosphorous removal occurs during AWT and is apparently due to incomplete settling of the floculant material generated by the chemical reaction. Soluble phosphorous tied up in the unsettled floc appears as non-soluble phosphorous in the laboratory analysis. Part of the soluble phosphorous removal rate reported is attributable to this apparent increase in the non-soluble fraction.

(7) The 1984 field investigation indicated influent WWTP total phosphorous concentrations for all flow conditions monitored to be 1.55 mg/l. Wet weather samples at the WWTP grit chamber indicated a weighted average of 1.3 mg/l although "first flush" values were as high as 6 mg/l. Wet weather total phosphorous concentrations noted at the Weiss Street Pump Station averaged 1.74 mg/l.

Interceptor Sewer Data

(8) The junction of the West Side Interceptor river crossing and the East Side Interceptor was identified as a hydraulic bottleneck. The Weiss Street Pumping Station relieves backwater/surcharge at the crossing and together with the Weiss Street gravity overflow constitute the dominant combined sewer overflow in the system.

(9) The intercepting system's major hydraulic bottleneck is the river crossing. Application of additional head to the crossing by raising the weir level at the Weiss Street Pump station would increase flows across the river. A five foot weir height increase was estimated to be optimum. Further increases provide diminishing marginal gains as flows are increasingly diverted from Weiss Street to other overflow points rather than to the WWTP.

Regulator Data

(10) Regulators (earlier modifed as part of the Hancock Street Detention Facility improvements (mid 1970's)) had been altered to cause the underflow orifices to close rapidly, limiting flows to the interceptor and thus committing the "first flush" of a rain event to in-line storage created at the chamber. EDP considered it more advisable to maintain higher underflow discharges to the interceptor, limited only to what the interceptor can maximally handle. Operation in this manner will commit the "first flush" of a rain event, generally carrying the highest concentration of pollutants, to the interceptor, and eventually to the WWTP.

(11) It appeared during the EDP inspection program that the mechanical float-operated mechanisms used to regulate flow at the regulation chambers were slow in responding to rising water levels, and it was speculated that a number of the float activated underflow orifices do not appreciably close during high water conditions. The City reported that the devices required constant surveillance to maintain operation.

(12) New vortex flow throttling devices having no-moving parts, large aperture openings (in comparison to conventional orifices) constructed of durable material (stainless steel) and reported to be nearly clog-free were envisioned to replace the existing mechanical float operated mechanisms in the regulators. These new vortex flow throttling devices combine conventional orifice flow (high discharge with low head) with vortex braking operation (low discharge with high head), thus replicating and enhancing the flow characteristic produced by the variable-orifice opening mechanical float-operated mechanisms.

In System Storage Data

(13) Potential in-line storage capacity in the Saginaw sewer network is extensive. A total storage volume of 3.93 million cubic feet (29.4 million gallons) has been estimated as available without threatening basement flooding. Regulator modifications and in-line storage devices recommended to achieve this in-line storage potential are inexpensive, and will be less costly to maintain than present float-type regulating devices.

(14) Regulators on the West Side are typified by conditions where the influent trunk sewers are on a steep incline to the regulation chambers which then level- off not far upstream. Far greater storage volumes could be generated for these trunk sewers if additional flow control structures were located upstream where the invert slopes are less steeply inclined.

B. CSO Management Model

(15) The Saginaw CSO Management Model, developed by EDP, is an inexpensive and flexible tool for preliminary planning and sizing of CSO facilities. The model provided a means by which the CSO problem could be analyzed and introduced time and probability to the quantity, frequency and duration of overflow events by using actual rainfall data collected over a long period of time.

(16) Modification of the original model developed in 1980 (Phase 1) using extensive field data collected in 1983 (Phase 2) and 1984 (Phase 3) has resulted in a fine-tuned edition which produces flow and water quality data closely matching field observations.

(17) During Phase 1 it was estimated that 25.6 percent of all wet weather flow would arrive at the WWTP and implementation of the partial and complete BMP programs would increase this percentage by factors of 1.31 and 1.93, respectively.

(18) Using the model as calibrated (based on field investigation performed in 1984) it is currently estimated that 37.7% of wet weather flow would arrive at the WWTP and will increase by factors of 1.35 and 1.91 for the reduced and complete BMP plans, respectively.

C. Implementation of Reduced BMP Plan

(19) Partial implementation of the reduced BMP plan including modifications to 12 of 35 combined sewer regulators, construction of one upstream in-system storage structure and increasing the

effective head at the Weiss Street crossover was completed for the construction price of $315,000.

(20) Modification of regulator chambers was accomplished under controlled conditions such that no dry weather sewage or wet weather combined sewage impacted the river during construction.

(21) Conditions within regulators were controlled such that no debris generated as part of the modification program impacted the river or interceptor sewer lines.

(22) Existing chambers were modified using concrete fill to eliminate areas of debris accumulation and resulting odors.

(23) Vortex valve flow regulators were constructed and installed in variable arrangements to conform to existing conditions within the regulator chambers. The devices were segmented for ease of installation particularly where access space is limited.

(24) Modification of regulator chambers was easily accomplished using standard construction practices particularly saw- cutting of concrete, core drilling and concrete forming and pouring.

D. Evaluation of Implemented BMP Plan

(25) With the exception of limited pollutant removal at the Hancock Street Facility, all reduction of CSO impact to the Saginaw River is attributable to increased in-system transient storage coupled with increased flow and "first flush" materials to the WWTP. Prior to this program, it is estimated that on an annual basis, 37.7% of all wet weather related flow would be handled by the WWTP.

(26) Implementation of the reduced BMP plan is estimated to have increased the percentage of total annual wet weather flow directed to the WWTP by 14.6% (from a previous "as is" level of 37.7%). Implementation of the remaining elements of the complete BMP plan could produce an overall incremental increase of 30% (increasing the total wet weather flow treated at the WWTP to 67.7%). For "Low Rainfall" events (less than 0.5 inches rain), the percentages of increase flow to the WWTP due to implementa-

tion of the reduced BMP plan and the complete BMP plan are estimated as 19% and 34.6%, respectively. For "High Rainfall" events (greater than 0.5 rainfall), the respective percentage increases are 10.5% and 21.8%. Table 2.1 presents estimated related increase of total systemwide wet weather discharge to the WWTP for different rainfall conditions as a result of the reduced and complete BMP plans.

(27) Implementation of the reduced BMP plan is estimated to have incrementally reduced (beyond "as is" levels) suspended solids loadings to the Saginaw River by 16%, BOD by 20% and total phosphorous by 8.5%. During periods of AWT (pickling liquor addition) at the WWTP, total phosphorous removals will increase 12.6% above pre-implementation levels. The rain intensity/percentage increase relationship noted for flow to the WWTP is consistent for suspended solids, BOD and phosphorous reductions for the Saginaw River. The greatest increase in reductions occurs during low intensity events with relatively lesser reductions udner high intensity conditions. Table 2.1 indicates the annual percentage reduction of total systemwide suspended solids and BOD loadings to the Saginaw River.

(28) On a weight basis, the reduced BMP plan implementation is estimated to have reduced wet weather suspended solid pollutant loads to the river by 1.4 million pounds per year and BOD by 0.47 million pounds per year and total phosphorous by 3000 pounds per year. Wet weather related total phosphorous loadings are estimated to be reduced by 3000 to 4000 lbs. per year depending on application of AWT .

(29) Incremental TSS and BOD removals of total wet weather loadings to river attributable to implementation of the complete BMP program (when completed) are estimated to equal about 29% and 30%, respectively. Total phosphorous reduction with secondary treatment is estimated to equal 16.7% and with AWT would equal 25% above pre BMP conditions. Mass removals due to implementation of the complete BMP plan (when completed) are estimated as 2.5 million pounds of SS, 0.68 million pounds of BOD and 7000 to 10,000 pounds of total phosphorous (latter estimate when AWT used). Consistent with the data for the reduced BMP plan, increased reductions of loadings to the Saginaw River were greater under low intensity rain conditions.

(30) WWTP TSS and BOD removal levels noted during the post construction evaluation period (late 1984) have not diminished, confirming early Phase 1 and Phase 2 projections.

(31) Figure 2.1 graphically illustrates the relative annual wet weather phosphorous loadings to the Saginaw River based on various treatment scenarios. Two sets of bar graphs were used to

TABLE 2.1

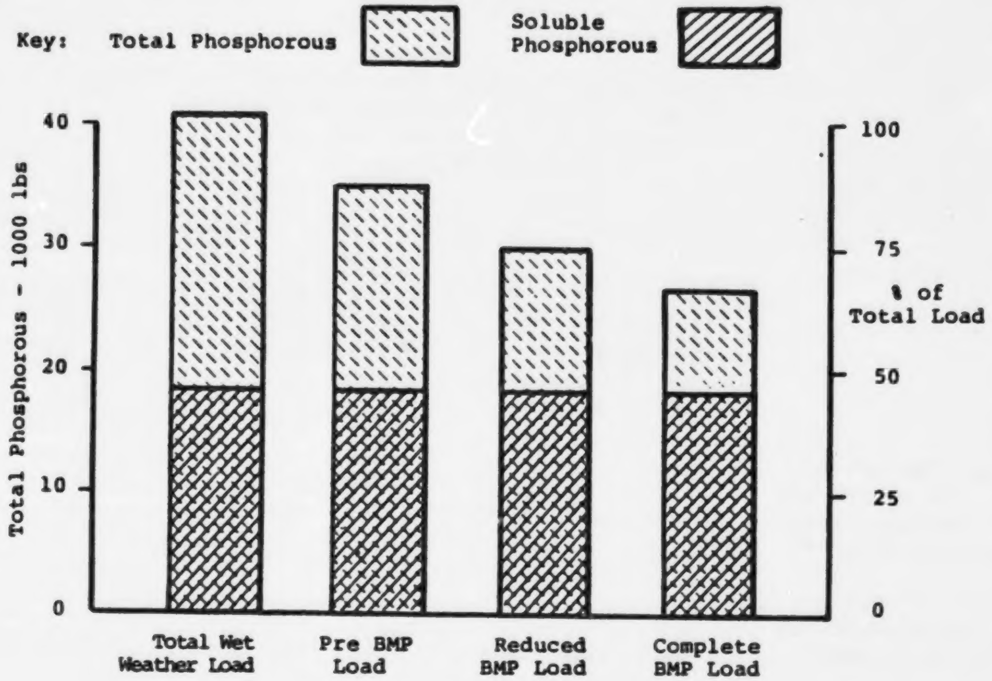
Pollution Abatement Effectiveness
of Best MAnagement Practices Program
Suspended Solids and BOD Emissions
Saginaw, Michigan

	Pre Project Condition			Results of Reduced BMP Program Implementation			Projected Complete BMP Program		
	L	H	T	L	H	T	L	H	T
A. Annual Percentage of Total Systemwide Wet Weather Sewage Discharge to WWTP	53.0	28.5	37.7	72.0	39.0	52.3	87.6	50.3	67.7
B. Annual Percentage of Suspended Solids Loadings to WWTP	50.0	26.0	35.6	72.0	38.0	52.3	91.0	48.0	66.0
C. Annual Percent Reduction of Total Systemwide Wet Weather Suspended Solids Loadings to Saginaw River	46.5	24.2	33.1	67.0	35.3	48.7	84.6	44.6	61.4
D. Annual Percent Reduction of Total Systemwide Wet Weather Sewage BOD Loadings to Saginaw River	53.4	27.6	36.6	67.6	41.8	57.0	77.4	50.0	66.7
E. Estimated Capital Cost (millions)		-							

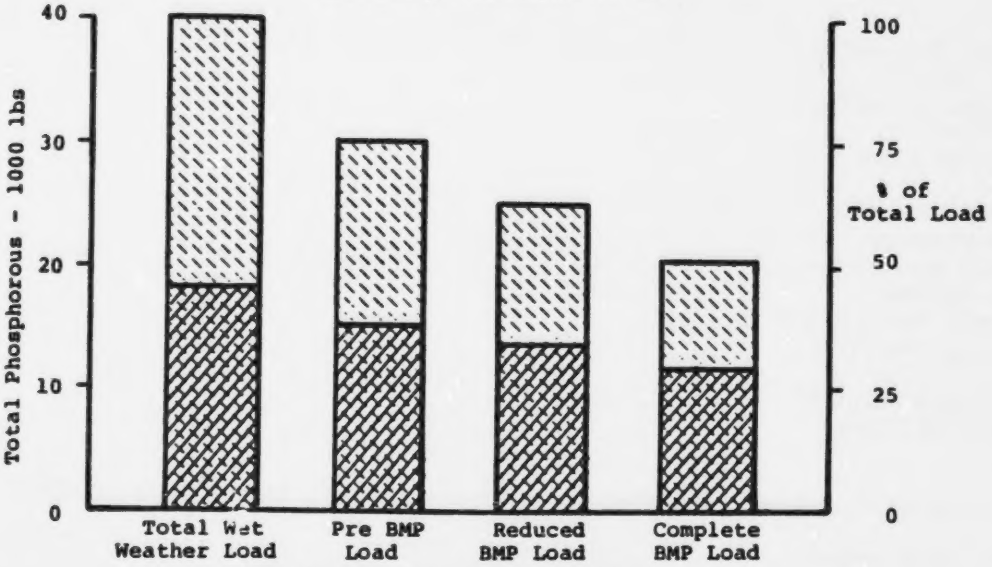
L = Low Accumulation Rain Event (0.5" or less)

H = High Accumulation Rain Event (greater than 0.5")

T = All Events (1978 Rainfall)



Secondary Treatment at WWTP



AWT Tertiary Treatment at WWTP

Figure 2.1 Wet Weather Total Phosphorous Loading to Saginaw River

indicate estimated removals with the WWTP providing secondary (top of figure) and AWT (bottom of figure). In addition, the soluble fraction of the total phosphorous is illustrated. Figure 2.1 clearly illustrates substantial reductions in total phosphorous loadings to the river due to treatment at the WWTP and that implementation of BMP measures greatly enhance total removals. Soluble phosphorous removal is non-existent with the WWTP providing only secondary treatment. Based on an annual total wet weather load of 41,000 lbs., the total estimated phosphorous loadings to the river would be 31,000 lbs. for the reduced BMP plan and 27,600 lbs. for the complete BMP plan with secondary (50%) treatment at the WWTP. The entire decrease is due to reduction of the non-soluble phosphorous fraction. With AWT (75%) treatment, residual loads of total phosphorous to the Saginaw River would be 26,000 lbs. and 21,000 lbs. for the reduced and complete BMP plans, respectively.

(32) Under optimum conditions of interceptor flow (low wet well, no upstream East Side flow) the maximum river crossover rates would increase from 53 to 73 cfs due to the Weiss Street Pump Station weir modification. Under the condition of high WWTP wet well and no upstream East Side flow, the maximum crossover rates would increase from 25 to 47 cfs due to program modifications. Upstream East Side Interceptor flow will always exist and this flow will induce greater head loss and surcharging of the interceptor further reducing the potential crossover flow. It is estimated that the modification of the Weiss Street Station spillover weir level has increased the crossover rate by about 20 cfs under all conditions and that operation of the WWTP wet well within the alarm range can handle this range.

(33) The WWTP currently operates in a near optimum manner during wet weather conditions. Future modification of the WWTP hydraulic conveyance system between major treatment elements could increase peak pumpage rates and reduce bypass.

(34) Roughly 500,000 cubic feet of off-line storage is used at the WWTP. Two aeration basins are used to store combined sewage delivered to the plant at rates limited by the WWTP conveyance capacity at the grit chamber where bypass occurs. Future modifications to increase hydraulic capacity may enable a greater portion of "first flush" materials to be stored in the aeration basins prior to bypass at the grit chamber.

(35) WWTP influent is pumped from a wet well to the grit chamber using 6 lift pumps. Pump combinations in use at any time are manually selected to maintain proper wet well elevations. Wet well alarms exist for high and low conditions but operation outside the alarm range is possible. Original

design drawings indicate a design wet well level 5 feet below the present low alarm level. Although pump operation at the design wet well level will increase wet weather river crossover rates, it may cause pump cavitation for certain pump combinations.

(36) With the exception of a few minor reported incidences of large stick debris cloggage, the flow and solids handling performance of the new vortex valve flow throttling devices replacing the mechanical float-operated controllers within the regulation chambers was satisfactory. No adverse flow backup or sedimentation problems were reported. On several occasions the devices were visually observed to pass "first flush" and scoured solids on draindown. Limited flow calibration measurements indicated that the devices generally operated as per manufacturer´s specifications.

(37) The performance of the new in-line control chamber at Salt & Vermont Streets which generated at maximum capacity a transient storage volume of 175,000 cubic feet was satisfactory. The new vortex valve flow throttling device which controlled the wet weather discharge from this facility as well as permitting passage of normal dry weather sewage worked extremely well. No blockages, sediment buildup nor adverse backups were reported. Minor odor problems with organic sediment buildup within "dead corners" of the in-line control chamber were noted and are attributable to incorrectly sloped concrete fillets placed during construction. This minor problem has since been corrected.

F. Optimization and the Recommended Facility Plan (Future Actions)

(38) Runoff simulation studies indicate that the most singularly effective measure for handling CSO in Saginaw is the inclusion of Best Management Practices (BMP). The BMP program consists of extensive in-line storage and system modifications to increase transient in-line storage and system hydraulic capacity so that the WWTP can handle more wet weather combined sewage.

(39) After implementation of the complete BMP plan, residual overflows will remain at six locations: Weiss Pump Station, Weiss Street gravity line, Emerson Pump Station, Fourteenth Street Pump Station, Webber Street Pump Station and Fraser Street (at the Queen Street/Fraser Street regulator). CSO treatment is needed to treat these overflows remaining after the BMP plan implementation. The most cost-effective program of CSO abatement was developed using the swirl concentrator technology at these locations. A non-linear optimization model based on swirl concentrator suspended solids removal was used as an aid to identifying optimal abatement configurations.

(40) Disinfection using sequential chlorine/chlorine dioxide dosage rates of 8 ppm/2 ppm for a total contact of 2 minutes is recommended, as required to maintain bacteria pollution in the Saginaw River at acceptable levels and would tend to minimize the production of carcinogenic materials associated with conventional chlorine application.

(41) The design maximum solids loading capacity at the WWTP plant is estimated to be exceeded during high rain events (greater than 0.5 inch rain) if the entire systemwide swirl concentrator program is implemented. A swirl degritter would then be recommended at the Weiss Street treatment complex to handle solids generated from the swirl treatment units pending a more detailed hydraulic investigation of the crossover as well as the grit removal capacity during high flows at the WWTP. Removal of solids at Weiss Street using the swirl degritter (followed by landfill disposal) will maintain acceptable solids loadings at the WWTP and will eliminate any potential problem of solids deposition at the river crossing.

F. Cost of the Recommended Facility Plan (Future Actions)

(42) Cost estimates are based on 1985 dollars (ENR = 4500). Capital costs for the recommended CSO Facility program are estimated to be 23.7 million dollars. The recommended CSO Facility plan would use swirl concentrator/swirl degritter technology, and the BMP in-line storage and system modification plan. The costs do not include the presently operating Hancock Street CSO facility (capital cost of $10.4 million, ENR=4500), nor the cost of land acquisition for the new swirl facilities.

G. Effects of Facility Plan Implementation On River Quality (Exerpted from EDP Facility Plan Report (2))

(43) Available storm-related water quality of the Saginaw River includes limited past storm-related data, an extensive one-dimensional hydrologic and ecological model of the river and bay, and recent data collected to characterize the runoff and impact. These data sources and previous analyses were reviewed and analyzed.

(44) CSO has been identified as a major contributor of the dissolved oxygen deficits in the Saginaw River following storm events. Dissolved oxygen was reduced from above 6 mg/l to as low as 4 mg/l as a result of CSO from a storm event with 0.83 inch total accumulation with typical river flow conditions. Under more critical conditions, the dissolved oxygen deficits will be greater.

(45) Sediment pollution has been identified as a major problem in the Saginaw River. CSO is a contributor to sediment pollution.

(46) Suspended materials have been identified as contributors to dissolved oxygen deficits by reducing light penetration and algal productivity. Suspended solids also contribute to aesthetically objectionable turbidity. Suspended solids may be carriers of other pollutants such as toxic materials and algal nutrients.

(47) CSO has been identified as a source of aesthetic pollutants including floatable pollutants and suspended solids that increase turbidity.

(48) A dissolved oxygen model based on the classical Streeter-Phelps equation was determined sufficient to model CSO dissolved oxygen impacts on the Saginaw River. DO levels as low as 4 mg/l have been predicted to be common as a result of CSO loadings.

(49) Simulation models have been successfully developed and calibrated for predicting the operation of the Saginaw combined sewer system and the Saginaw River water quality response. Overflow volumes and quality in terms of suspended solids and five day BOD can be predicted on an hourly basis. Dissolved oxygen in the Saginaw River can be predicted in response to CSO pollutant loadings. The capability of testing CSO abatement alternatives has been incorporated in the models.

(50) Nonpoint source loadings resulting from combined sewer overflows in the City of Saginaw are significant contributions to pollutant loadings to the Saginaw River. CSO loadings were estimated using a long term simulation model to be 8,600,000 pounds TSS and 2,300,000 pounds BOD for the calendar year of 1977 consisting of 78 storms creating overflows.

(51) Implementation of the recommended CSO Facility program would raise the dissolved oxygen level from 3.2 mg/l to 4.4 mg/l for a 1.9 inch storm impacting July 8, 1980 river conditions. Dissolved oxygen gains for less intense events would be smaller. Significant protection from excessive dissolved oxygen deficits would occur primarily for severe rain events at or above 0.8 inches of total accumulation with significant rain intensity.

(52) The recommended control plan would substantially eliminate the sediment pollution component caused by CSO. Settleable solids would be over 90% reduced.

(53) Over half the wet weather related suspended solids emmisions would be eliminated. The suspended solids reduction will aid dissolved oxygen levels by increasing light penetration and subsequently algal productivity, and by lowering algal

nutrients associated with suspended solids. Lowered turbidity levels will also increase the aesthetic appeal of the river water.

(54) Pollutants that can be associated with the removed solids in CSO will also be removed. These pollutants include toxic pollutants such as heavy metals and algal nutrients.

(55) Aesthetic qualities of the river will be significantly improved increasing the enjoyment of the river for recreational purposes. Floatables and suspended materials will be reduced.

CHAPTER 3

RECOMMENDATIONS

The conclusions stated in Chapter 2 of this report clearly indicate the effective use of elements of the BMP plan in reducing CSO to the Saginaw River. To date, the entire BMP plan has not been scheduled for completion as the the funding for implementation of complete facility plan elements is not available. In addition, the 1984 investigation of the system hydraulics during the construction and post evaluation phases indicated areas of system operation where additional review and "fine-tuning" may be necessary in the future due to hydraulic changes induced by implemented elements of the BMP plan. The following recommendations of this study for the City of Saginaw's sewerage system are presented in order of importance:

1) The reduced BMP plan included modification of only 12 of 35 regulator chambers. All other regulation chambers should be updated to include new vortex flow regulating devices and appurtenances designed to maximize storage and minimize bypass to the river.

2) One of 14 proposed upstream in-line storage chambers was completed. Facilities for the remaining locations should be designed and constructed. The recommended order of priority for implementation of these locations is as follows:

In-line Storage Priority List

1. River & Cambrey Streets
2. Birch & Harris Streets
3. Gratiot & Michigan Avenue
4. Cass & Woodbridge Streets
5. Hamilton & Williams Streets
6. Union Avenue & Delaware Street
7. Jackson & Mason Streets
8. MacKinaw Street & Michigan Avenue
9. Troy & Boxwood Streets
10. 17th & Perkins Streets
11. 14th & Norman Streets
12. Hayes Street & Michigan Avenue
13. Weiss & Delaware Streets

The in-line storage facility at Weiss Street and Delaware Street should be completed as part of the CSO treatment facility implementation at the Weiss Street Pump Station.

3) Existing gate opening and pump "turn-on" sequencing at the Hancock Street Pump Station should be reviewed to ascertain whether the operation is optimal. The existing monitoring equipment at the Hancock Street Pump Station should be upgraded, if necessary, for the purpose of monitoring station operation in compliance with optimization recommendations.

4) The pump initialization equipment at the Weiss Street Pump Station should be reviewed to optimize turn-on/shut-off elevations and to ascertain whether supplemental operating controls are necessary due to the weir modification performed as part of reduced BMP plan.

5) The operations of the Hancock Street Facility and the Weiss Street Pump Station should be reviewed to ensure that the Hancock Street Station operates prior to the Weiss Street Station during a storm event. Detention storage/treatment currently available at Hancock Street will thus be utilized prior to untreated bypass at Weiss Street (for areawide storm events).

6) The CSO control facility for the Weiss Street area should be implemented as overflows at this location are still the dominant source of wet weather pollutant load to the Saginaw River.

7) Under extremely high rate conditions, wet weather flow pumped to the WWTP grit chamber is split with partial overflow to the WWTP bypass. Overflow commences at a total rate less than the existing WWPT flow-through capacity. Possible replacement of the WWTP overflow bypass weirs with sluice gates may allow initial storm flow containing "first flush" materials to pass to the aeration basins for storage. Later cleaner flows would be bypassed, if necessary due to full basin conditions, by opening of the sluice gates.

8) Currently potential pump cavitation conditions prevent maintaining WWTP wet well levels at the initial design elevation. Although the City WWTP personnel maintain maximum wet weather throughput whenever possible to the WWTP, the elevations currently maintained, reduce the hydraulic capacity of the interceptor system during major events causing overflow at upstream pump stations. Hydraulic conditions of influent pump intakes at the WWTP wet well should be investigated to determine if baffles, spoilers or intake vanes could reduce cavitation and allow lower operating levels, and result in lesser upstream overflow magnitudes.

9) Evaluation of the WWTP layout for potential hydrualic bottlenecks at maximum capacity should be performed. In some areas, piping systems are believed to be the limiting factor in

plant operations as opposed to the performance capability of treatment processes. This investigation should ascertain whether any changes to increase flow rate would improve overall WWTP wet weather capacity/reduce bypass and whether the water quality benefits that may result would be cost effective.

CHAPTER 4

DESCRIPTION OF CITY OF SAGINAW SEWERAGE SYSTEM

4.1 Descriptive Background Material

Saginaw is located in the eastern part of central Michigan north of Detroit, as shown in Figure 1.1. The Saginaw River is a short section of a major drainage area leading to Saginaw Bay, Lake Huron. Saginaw is located at the start of the river and on both banks for about a five mile stretch. The Tittabawassee, Shiawassee and Cass Rivers are tributary to the Saginaw River just upstream of the City. The topography of the area is flat, and the elevation is approximately 600 ft. The Saginaw River divides the 10,437 acre area of the City of Saginaw into an East Side section having an area of 5,634 acres, and a West Side section of 4,803 acres. The Saginaw River drainage basin above the City has an area of 6,200 sq. mi. The average daily stream flow is estimated at 10,000 cfs with miniumum daily flow of 500 cfs. Flood flows of 62,000 cfs have been observed.

The entire river is only about 22 miles in length and has a gradient of about one inch per mile or less along its entire length. The river is used extensively as a commercial waterway, and it is regularly dredged by the Corps of Engineers to maintain the shipping channel. The depth of this channel ranges from 16.5 feet at Green Point to 27 feet at river mouth. There are five dredged turning basins along the length of the river. The width of the Saginaw River ranges from 365 feet at Green Point to 1,700 feet near the Middle Grounds. Winds significantly affect the water level in Saginaw Bay and in turn this water level strongly affects the direction of flow in the Saginaw River. Sustained southwesterly winds lower the level of the bay and temporarily increase the river's velocity discharge. Sustained northeasterly winds, on the other hand, raise the water level in the bay and decrease stream velocity and discharge, often reversing the flow of the river.

The area which is surrounded by the Great Lakes, has a quasi-marine environment. Temperatures stay warmer later in the year and cooler during the spring because of the influence of the large water masses. Saginaw's normal annual precipitation is about 28.55 in., over 50% of which falls during the summer between March and September. Summer storms occur as showers and thunderstorms. Most of the winter precipitation is snow which accumulates until the spring thaw. Beginning in March or April, melting snow combined with rainfall can produce high overflow volumes and increased discharge of pollutant loadings to the river. Most of the storms that occur are of low average intensity. Over 90% of the time, the average intensity of rainfall which produces runoff is less that 0.10 in./h.

The City of Saginaw is serviced by a combined sewer system. Combined sewer overflows during rain events are a significant contributor of pollutants to the Saginaw River. Overflows may occur at 35 regulator chambers during increased flow events. Overflows may also occur at the five pump stations designed to relieve the system during flooding conditions. Overflows occur when the flow in the combined collection system is greater than three to six times the dry weather flow rate depending upon the regulator location. It is estimated that over 60 overflows occur per year.

Impacts on the Saginaw River from CSO include dissolved oxygen depletion in the downstream segments of the river and increased bacteria levels. Public health impacts can be of equal importance to the organic loading to the river, because of the high level of micro-organisms associated with combined sewer overflows. Suspended solids and floatable material also create visual problems and further aggrevate the continual need to dredge the channel for maintaining commercial navigation patterns.

4.2 Sewerage System Overview

Saginaw is serviced by a combined sewerage system. The overall service area is depicted in Figure 4.1. Catchment area boundaries of individual subsystems considered in this study along with proximate location of regulators are shown in Figure 4.2.

An intercepting sewer shown in Figure 4.3 runs along both banks of the Saginaw River, eventually joining on the East Side and running to an advanced secondary WWTP. The West Side interceptor crosses the river at Weiss Street to join the East Side Interceptor, which continues to the WWTP at the northeast limits of the City. A portion of the Saginaw township outside the city limits is also serviced by the combined sewer system with dry weather flow joining the Saginaw intercepting sewer system and eventually treated at the WWTP. There are also a number of areas within Saginaw whose stormwater runoff is not diverted to the Saginaw combined sewer system. State Highway I-75 with downtown connectors covers an area of about 100 acres and has its own drainage system. A large newly developed residential area in the southeast section of the City has a separated storm sewer/sanitary sewer system. A number of industrial and commercial areas near the river are drained to the river and are independent of the combined sewer system.

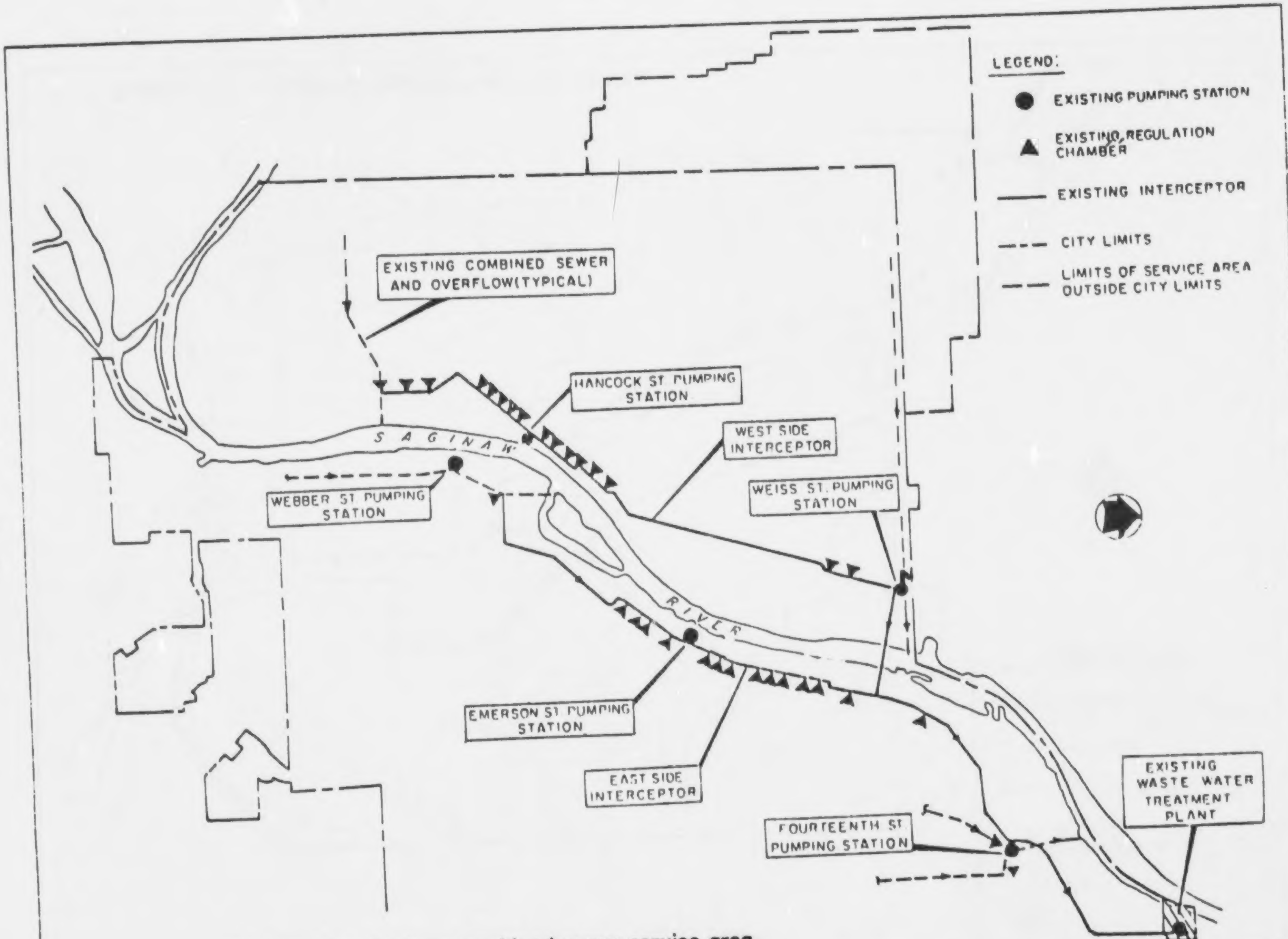

LEGEND:
● EXISTING PUMPING STATION
▲ EXISTING REGULATION CHAMBER
EXISTING INTERCEPTOR
CITY LIMITS
LIMITS OF SERVICE AREA OUTSIDE CITY LIMITS
EXISTING COMBINED SEWER AND OVERFLOW(TYPICAL)
HANCOCK ST PUMPING STATION
WEST SIDE INTERCEPTOR
WEISS ST. PUMPING STATION
WEBBER ST PUMPING STATION
S A G I N A W
R I V E R
EMERSON ST PUMPING STATION
EAST SIDE INTERCEPTOR
FOURTEENTH ST PUMPING STATION
EXISTING WASTE WATER TREATMENT PLANT

Figure 4.1 Saginaw combined sewer service area.

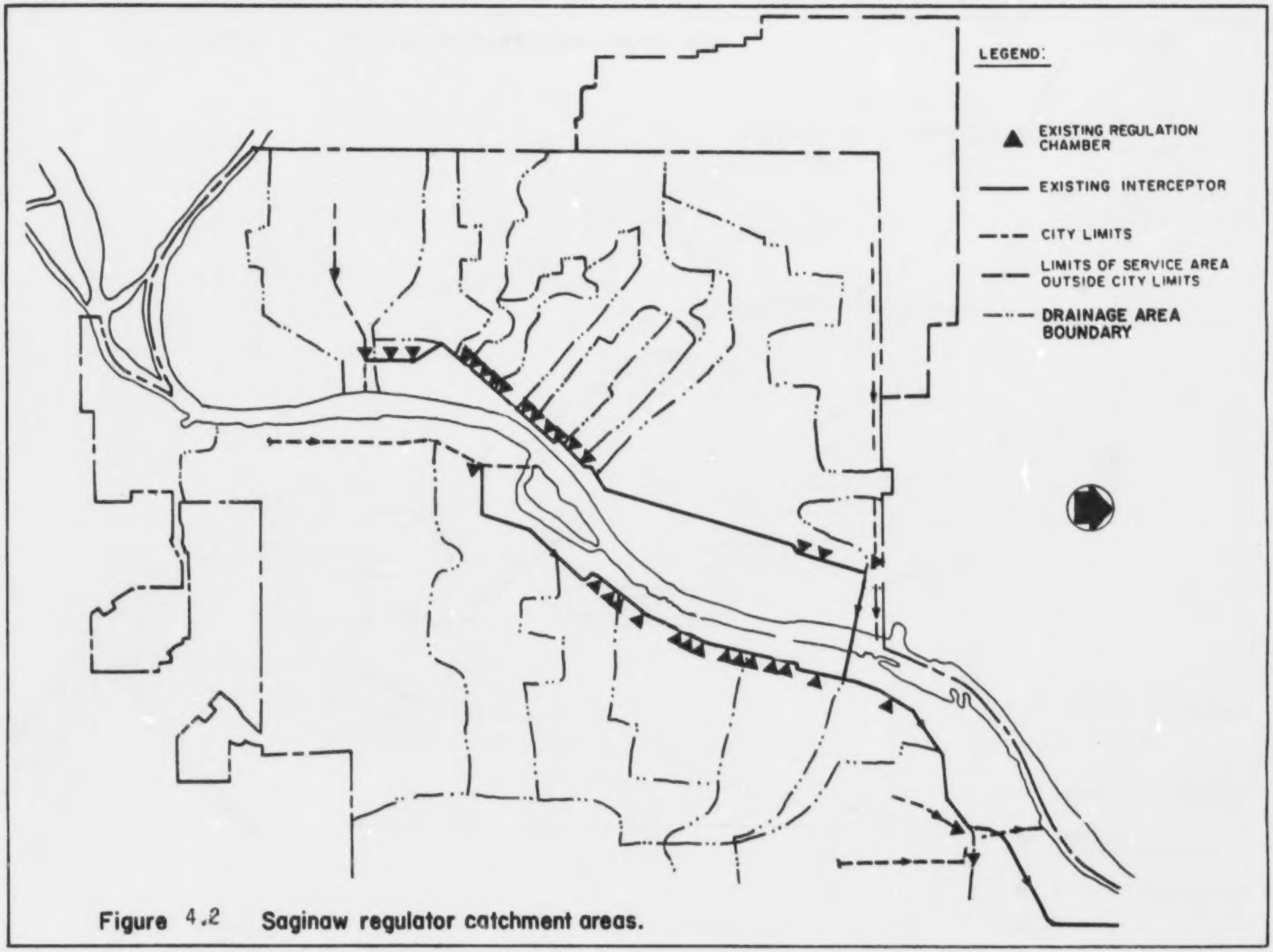

LEGEND:
EXISTING REGULATION CHAMBER
EXISTING INTERCEPTOR
CITY LIMITS
LIMITS OF SERVICE AREA OUTSIDE CITY LIMITS
DRAINAGE AREA BOUNDARY

Figure 4.2 Saginaw regulator catchment areas.

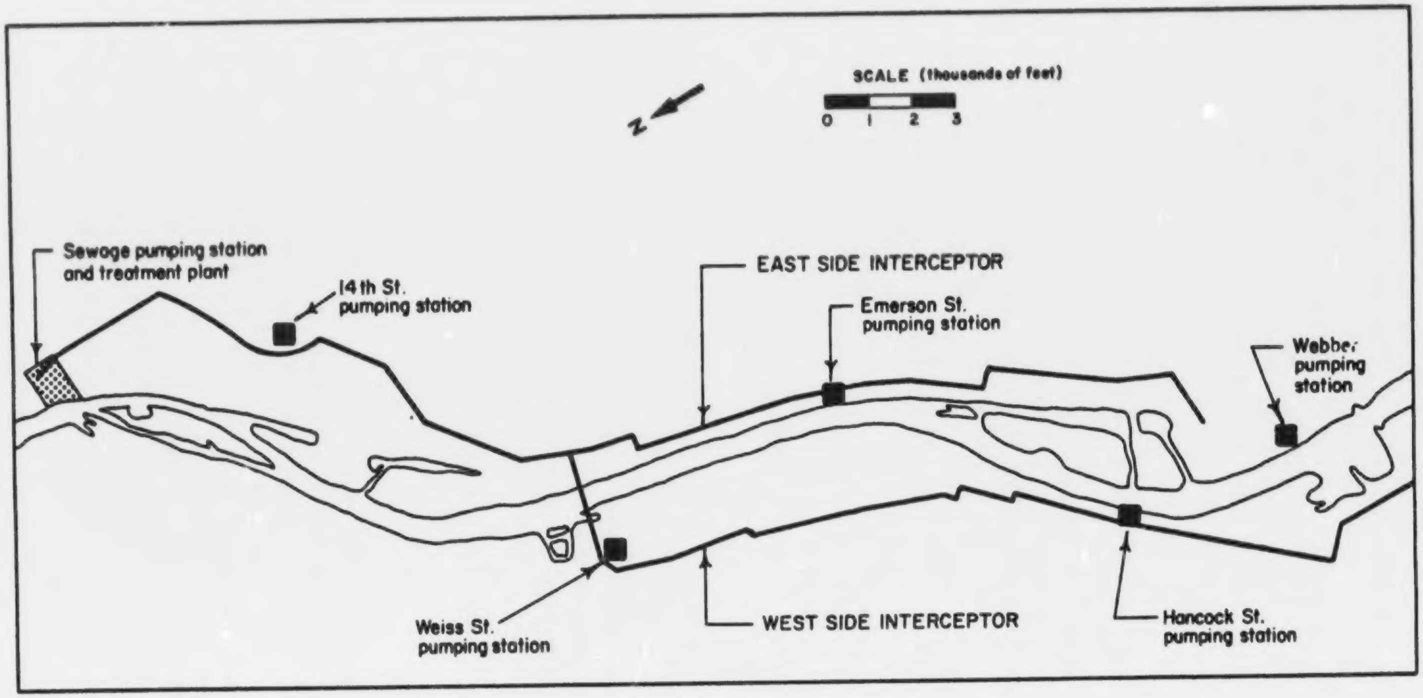

SCALE (thousands of feet)
0 1 2 3
N
Sewage pumping station and treatment plant
14th St. pumping station
EAST SIDE INTERCEPTOR
Emerson St. pumping station
Webber pumping station
Weiss St. pumping station
WEST SIDE INTERCEPTOR
Hancock St. pumping station

Figure 4.3General plan of interceptor sewer system.

Because many of the sewers are below flood river stage, the problem of collecting sewage involves stormwater pumping during periods of high river level. Typically, flood stages can exist for periods up to two weeks, during which time rainfall producing runoff can occur. Flood stages of the Saginaw River have all occurred during the months of March, April and May. During flood river levels, storm flows will discharge over the regulator chamber weirs into the intercepting system. Backwater gates will prevent discharge of flood river water into the sewerage system.

Flood pumping stations located at Webber, Emerson, and Fourteenth Streets on the East Side, and Hancock and Weiss Streets on the West Side, pump either from the interceptor system or from tributary lines directly before the interceptor system. Stormwater and flood pumping stations are of circular caisson type construction. The station at Hancock Street has eight electric-driven pumps, and the one at Weiss Street has three electric-driven pumps. On the East Side the station at Webber Street has three electric-driven pumps; the Emerson Street Station has six gasoline engine-driven pumps; and the Fourteenth Street Station has three electric-driven pumps. All pumps are similar, each having a fixed capacity of 100 cfs, those at Webber Street and Fourteenth Street operate at 15-ft head, and the remainder at 28-ft head.

The interceptor system operates at hydraulic gradients independent of the invert grade and has sufficient slope to convey the stormwater laterally to the pumping stations. The intercepting sewers thus serve the dual purpose of collecting sewage and carrying storm flows during flood river levels. The interceptor system consists of tunnel sewers ranging from 42 to 78 inches in diameter. The tunnels are approximately 40 feet below ground level.

4.3 WWTP Details

The City of Saginaw operates a 32.7 mgd secondary WWTP located on the east bank of the Saginaw River near the Saginaw-Buena Vista line. The plant receives wastewatwer from the City of Saginaw, portions of Saginaw, Spaulding and Bridgeport Townships, and from a portion of Buena Vista Township including the Saginaw Steering Gear complex. The City of Saginaw's sewer system conveys stormwater as well as municipal and industrial wastewater. In the early 1970's, secondary treatment facilities were added to the existing primary plant. By 1975 the secondary facilities were operational and in 1976 phosphorous removal was also provided.

Wastewater enters the plant in a 72-inch interceptor sewer. After screening and grit removal, the wastewater is settled in four primary sedimentation tanks. Primary effluent passes into the conventional activated sludge system where ferrous sulfate or waste pickle liquor can be added for phosphorus removal as necessary to meet effluent discharge standards. Final effluent is chlorinated before discharge to the Saginaw River.

The current NPDES permit for the plant requires that monthly average BOD and suspended solids concentrations not exceed 25 and 30 mg/L, respectively. Effluent total phosphorous concentrations must be less than 1 mg/L. In addition, the permit calls for fecal coliform concentrations less than 200 per 100 ml, and an effluent pH between 6.5 and 9.0.

A review of plant operating records indicates that the monthly average BOD and suspended solids concentrations are consistently below 10-15 mg/L and thus in compliance with the NPDES permit.

Prior to 1968 sewage discharging from the City of Saginaw was given primary treatment with chlorination. Average annual design rate was 32 cfs with a peak hourly capacity of 80 cfs. Wet well pumping capacity at the headworks totaled 182 cfs. Overflow weir settings at the regulators in the system were set to dischage untreated sewage mixed with stormwater at rates exceeding five times the dry weather flow. Regulator weir levels have been adjusted several times over the years in an effort to minimize the amount of untreated sewage.

The primary plant was then upgraded to secondary treatment including biological activated sludge with phosphorous removal. Although the modified plant is designed for a nominal flow of 50 cfs and an estimated maximum flow of 128 cfs, the secondary treatment unit operations are sized for about 160 cfs discharge. All storm flows entering the WWTP in excess of 108 cfs are split at the grit chamber with bypass receiving only chlorination. Currently, bypass beyond 108 cfs occurs at a spillover weir such that increasing influent flow rates results in simultaneous increased flow through the plant and increased bypass. The average dry weather flow is about 32 cfs.

All flow beyond the grit chambers receive primary treatment. There are four secondary aeration tanks following the primary clarifiers. Normally two aeration basins are used for dry weather flow and wet weather discharges up to about 84 cfs. The two other aeration basins are used as storage for wet weather discharges above 84 cfs. If these basins are filled, flow through conditions result. The storage contents after an event (or when influent rate significantly decreases) of these two tanks containing primary treated wastes (plus some additional detention) then receive full primary/secondary treatment as these volumes are pumped back to the headworks. Any flows beyond 84 cfs and in excess of the two aeration tank storage volume will receive primary treatment, limited aeration with phosphorous removal treatment (if necessary), secondary clarification and disinfection.

4.4 Regulator Chambers

The regulator chambers of interest are those points controlling discharge to the interceptor or to the river, from each of the tributary catchment areas. The simulation model described in Chapter 6 incorporates the operation of each regulator in the system. Physical operational details and dimensions were required such as orifice underflow openings, weir heights and float operations. Chamber measurements were compared with details given in the system plans, and missing values filled-in from the plans. If discrepencies occurred, the EDP field measured values were used in the analysis of regulator operation. Regulator details and modification details are presented in Chapter 7.

4.5 Hancock Street Storage/Treatment Facility

In 1977 the Hancock Street Flood Pumping Station was modified to include a 3.6 MG storage/treatment facility. The existing pumping station was upgraded to include electric-driven pumps. A multi-purpose underground storage/treatment facility with above ground municipal parking was constructed. In-line storage potential of the combined sewers in the Hancock Street area was increased by modifying the existing static regulators in the system. Modifications to the regulators included replacing the tide gates at the outfall with motor-operated sluice gates to increase the maximum storage capacity of the system. Before modification, only dry-weather flows and some wet-weather flows were diverted to the interceptor during normal river stages; now, significant flows are detained and diverted to the interceptor.

Over the last several years severe operational difficulties during intense rainstorms have arisen due to the motor-operated sluice gates in the Hancock Street catchment area. The sluice gates open or close in a finite period of time. If the gates do not open to permit rapidly developing peak flows to overflow, then severe surcharge problems will result. Replacement of these gates with fixed-opening weir structures was included in the construction project completed as part of the partial BMP plan.

During the storm events, combined flows diverted to the interceptor by the regulators activate the Hancock Street pumping station as the interceptor flow level increases. Sluice gates between the interceptor and the pumping station wet well open automatically and pumping is sequentially controlled by sensing the wet well water elevations. All flows to the storage/treatment basins are pumped. During extreme river flooding, the system will revert back to operation as a flood control pumping station, with flows to the basin being diverted to the river once the system has been filled to capacity.

Combined flows pumped to the Hancock Street storage/ treatment facilities sequentially fill a series of paired basins. The facilities operate under two types of storm flow conditions:

1. For certain flow volume storm events, the flow will be captured and totally stored, with subsequent release to the interceptor as capacity becomes available.

2. For storm events producing flow that exceeds the basin storage capacity, the system will store and treat the combined flow by sedimentation. The facility is designed to disinfect the overflow before discharge to the river.

4.6 Dry Weather Flow Characteristics

Formulation of the basis of design for CSO control facilities required input of data, with regard to the quality and quantity of dry weather flows, so that network models (See Chapter 6) could be calibrated. This section summarizes efforts to determine the quantity of dry weather flows at various points in the City of Saginaw system. Dry weather flow characteristics at the Saginaw WWTP are reviewed in section 4.6.1. Results of EDP measurement program are given in section 4.6.2. Estimates of dry weather flows per catchment area are given in section 4.6.3.

4.6.1 WWTP Dry Weather Flows

Dry weather flows were investigated at the treatment plant and throughout the system to establish a basis for simulating catchment area dry weather flows. The simulation program described in Chapter 6 required estimates of dry weather sewage contributions per catchment area for spring infiltration (non-rainfall) conditions as well as for non-spring, non-rainfall periods. These two estimates were necessary since the simulation model was geared to investigate rainfall/runoff conditions over a calendar year. The WWTP flow analysis was conducted for the period of 1978-1979. Flows at the plant were separated between dry days and wet days, and averaged (Table 4-1). The period was further divided between spring (April-June) and the other nine months to account for high groundwater infiltration conditions. The average dry day, non-spring flow was taken as the long-term dry weather flow average for the year including industrial contributions. The difference between the dry day non-spring average and the dry day spring average, 5.4 cfs, was considered a rough estimate of spring infiltration.

4.6.2 EDP Field Discharge Measurements

During October of 1979, EDP conducted a field investigation, including sewer system flow measurements. Flows were measured both for rain conditions caused by a small storm that occurred during the investigation period and for dry weather conditions throughout the system. Dry weather flows were measured at a number of the more important points throughout the system, with many of the measurements repeated at different times of the day for verification. The measured values were used to estimate flows at points where measurements were lacking to develop a complete distribution of dry weather flows throughout the system. Yield factors for known areas were calculated using the measured flow values and the corresponding catchment areas. For unknown neighboring areas similar land use yield factors were used to generate dry weather flow predictions based on catchment areas. All measured dry weather flows were checked using this same procedure of comparing similar or neighboring areas' yield factors.

Accumulations of both measured and predicted values in the interceptor were compared to measured values in the interceptor to check for closure of the various flow values. These results are presented in Table 4.2. From the field inspection program two areas of possible significant industrial contribution were identified, the foundry in the Fraser Street area on the West Side, and the lower industrialized area just south of the WWTP on the East Side. The differences between downstream flow measurements and upstream flow predictions and measurements were used to estimate the industrial contributions at these two points. A large industrial contribution of approximately 8.2 cfs was noted for the West Side Fraser area . A smaller contribution of 4.3 cfs was calculated for the East Side industrial area. Using the above industrial flow

TABLE 4.1

WWTP FLOWS, 1978-1979

	Flow Daily Average (cfs)		
Seasonal Periods	All Events	Wet Days	Dry Days
Spring (April-June)	46.9 85.6*	65.5 83.8*	42.8 86.2*
Non-Spring (Jan-March & July-Dec)	40.4 86.8*	56.0 86.5*	37.4 88.3*
All Seasons	42.2 86.4*	59.8 85.5*	39.4 87.8*

* BOD Removals

TABLE 4.2 SYSTEM DISTRIBUTION OF DRY WEATHER FLOW (cfs)

		Meas. Flow	Pre-dicted	Accum-ulative	Meas. cfs	Pre-dicted	Accum-ulative		
Weber	○	1.72			0.91			○	Fraser
Birch	○		1.93			8.30		△	Industrial
Holland	○	0.80			9.20		9.20	○	Dearborn
Atwater	■	3.50		3.65				■	Lee
McCoskry	○	1.01			0.24			○	Mackinaw
Holden	○		0.28					○	Van Buren
Emerson	○	0.46						○	Cass
Hoyt	○	0.06			1.41			○	Adams
Thompson	○	1.33						○	Court
Millard	○	0.12			12.60		11.4	⊡	Hancock
Janes	○	0.38				.05		○	Ames
Federal	○	0.44				.31		○	Hayes
Genesee	○	0.58				.24		○	Throop
Johnson	○	0.08						○	Miller
Fitzhugh	○	0.31				.24		○	Remington
Carlisle	○	0.72				.45		○	Genesee
Washington	■	9.40		9.56		.14		○	Cronk
							12.70	■	Carrollton
							15.90	■	Weiss
First	○	0.37			3.2			○	Weiss
Fifth	■	25.50		25.50					
16th Street	○	2.60							
	⊡								
14th Street	○		0.41						
Industrial	△		4.30						
WWTP	WWTP	32.5		32.5					

⊡ Pump Stations
■ Points on Interceptor
○ Regulator Points Before Interceptor
△ Industrial

estimates, a reasonable flow balance was achieved throughout the system. Measured flow values at various points in the interceptor and at the WWTP closely matched the accumulated dry weather and industrial contributions developed through the analysis of field inspection data.

4.6.3 Simulation Model Dry Weather Flows

As the field inspection program occurred in October, 1979 the dry weather flow analysis presented in Table 4.2 represents lower than average flow conditions typical of the fall. These results (excluding the industrial contribution) were scaled upwards to match the long-term average dry weather flow as calculated at the WWTP (See Table 4.1). The ratio of WWTP long-term average flow (non-spring) to the October, 1979 field program results (See Table 4.1) were used to scale all the dry weather flow values throughout the system. The same ratio was used with the spring WWTP flow average to complete the spring-time distribution. The results are summarized in Table 4.3. The non-spring and spring dry weather sewage estimates per catchment area were then used as inputs in the simulation model.

4.7 Wet Weather Flow Considerations

Wet weather flows in combined sewers consist of dry weather (sanitary) flow, infiltration, and storm-induced runoff. The storm-induced runoff includes flows from building connections, such as roof leaders; cellar, yard and foundation drains; catch basins; storm sewers; and surface runoff through manholes. This runoff is collected and transported to trunk sewers that convey flow to various diversion points. The diversion points regulate flow entering interceptor conduits, which carry the flow to treatment facilities. During a storm event, that portion of the flow which cannot enter the interceptor, overflows at the regulator connections to the Saginaw River.

Various data sources and analyses used to assemble the wet weather predictive features of the simulation model are described in this section. A discussion of rainfall analysis is presented in Section 4.7.1. Details of the 31 catchment areas used in the modeling analysis are presented in Section 4.7.2. Runoff parameters for the catchment areas are presented in Section 4.7.3. Details of the stormwater pumping stations are discussed in Section 4.7.4.

TABLE 4.3 SIMULATION MODEL DRY WEATHER FLOWS (cfs)

		Oct. Field Analy.	Model		Oct. Field Analy.	Model			
			Non-Spring	Spring		Non-Spring	Spring		
Weber	○	1.72	2.12	2.59	0.91	1.12	1.20	○	Fraser
Birch	○	1.93	2.37	2.91	8.20	8.20	8.20	○	Industrial
Holland	○	0.08	0.10	0.12	0	0	0	○	Dearborn
McCoskry	○	1.01	1.24	1.53	0.24	0.29	0.36	○	Mackinaw
Holden	○	0.28	0.34	0.42	0	0	0	○	Van Buren
Emerson	○	0.46	0.57	0.69	0	0	0	○	Cass
Emerson	▣				1.41	1.73	2.13	○	Adams
Hoyt	○	0.06	0.07	0.09	0.03	0.04	0.05	○	Court
Thompson	○	1.33	1.64	2.01				▣	Hancock
Millard	○	0.12	0.15	0.18	0.05	0.06	0.07	○	Ames
Janes	○	0.38	0.47	0.57	0.27	0.33	0.41	○	Hayes
Federal	○	0.44	0.54	0.66	0.21	0.26	0.32	○	Throop
Genesee	○	0.58	0.71	0.86	0.03	0.03	0.04	○	Miller
Johnson	○	0.08	0.10	0.12	0.21	0.26	0.32	○	Remington
Fitzhugh	○	0.31	0.38	0.47	0.39	0.48	0.59	○	Genesee
Carlisle	○	0.72	0.86	1.05	0.12	0.15	0.18	○	Cronk
								▣	Weiss
First	○	0.37	0.46	0.56	3.20	3.94	4.83	○	Weiss

16th Street	2.60	3.20	3.93	○
14th Street	0.41	0.50	0.62	○
Industrial	4.30	4.30	4.30	
WWTP	32.5	37.40	42.80	

WWTP

▣ Pump Stations

○ Regulators

4.7.1 Rainfall Analysis

Rainfall records were required as data input for the computer simulation work. A ten year period (1969-1978) of hourly recorded rainfall records at a number of stations in proximity to Saginaw were acquired from the National Weather Service. Daily rainfall records recorded at the Saginaw WWTP and the Tri-City airport (near Saginaw) were also acquired and examined in conjunction with the hourly analysis. Hourly rainfall information are collected in Saginaw.

Four stations recording hourly rainfall values were considered including Vassar - 20 miles east of Saginaw; Sebewaing and Beaverton - over 30 miles northwest and northeast, respectively; and Stanton - 50 miles west of Saginaw. Rainfall data collected at Vassar was selected for use in this study due to its proximity to Saginaw, and due to the lack of any major geographical features that may render Saginaw rain patterns substantially dissimilar from Vassar rain patterns. Sebewaing was not selected due to its location on Saginaw Bay and the likelihood of local weather anomalies (discussed further below). Stanton and Beaverton were not considered since their distances from Saginaw are greater than Vassar.

Standard procedures were followed to analyze the ten year period storm intensities and frequencies, and for total annual rainfall. A scatter diagram was developed to compare Saginaw rainfall as recorded daily at the WWTP to the Vassar rainfall records (See Figure 4.4). Distinct events were discerned from the Saginaw records using a decision rule of at least twenty four hours of low or no precipitation between major rain days. Rainfall was totalled for the resulting periods and compared to the totals for the corresponding periods in the Vassar records. All individual events for a period between 1975 and 1978 were plotted. Points lying further from the 45 degree equivalence line were noted to be primarily spring events when local variations in precipitation are more likely. The scatter diagram shown in Figure 4.4 suggests a reasonably good comparison between the two rain gages. Similarity in annual rainfall totals given in Table 4.4 for Vassar, Sebewaing and the Tri-City Airport in Saginaw further support this conclusion from the scatter diagram.

Since the prior CSO management study (1) in the Saginaw area had used rainfall records at Sebewaing as the basis for modeling Saginaw rainfall, Sebewaing rainfall characteristics were compared to Vassar and Tri-City Airport rainfall characteristics. The cumulative rainfall of the one-year design storm used to size the seven detention basins in the prior study equalled 1.9 inches of rainfall.

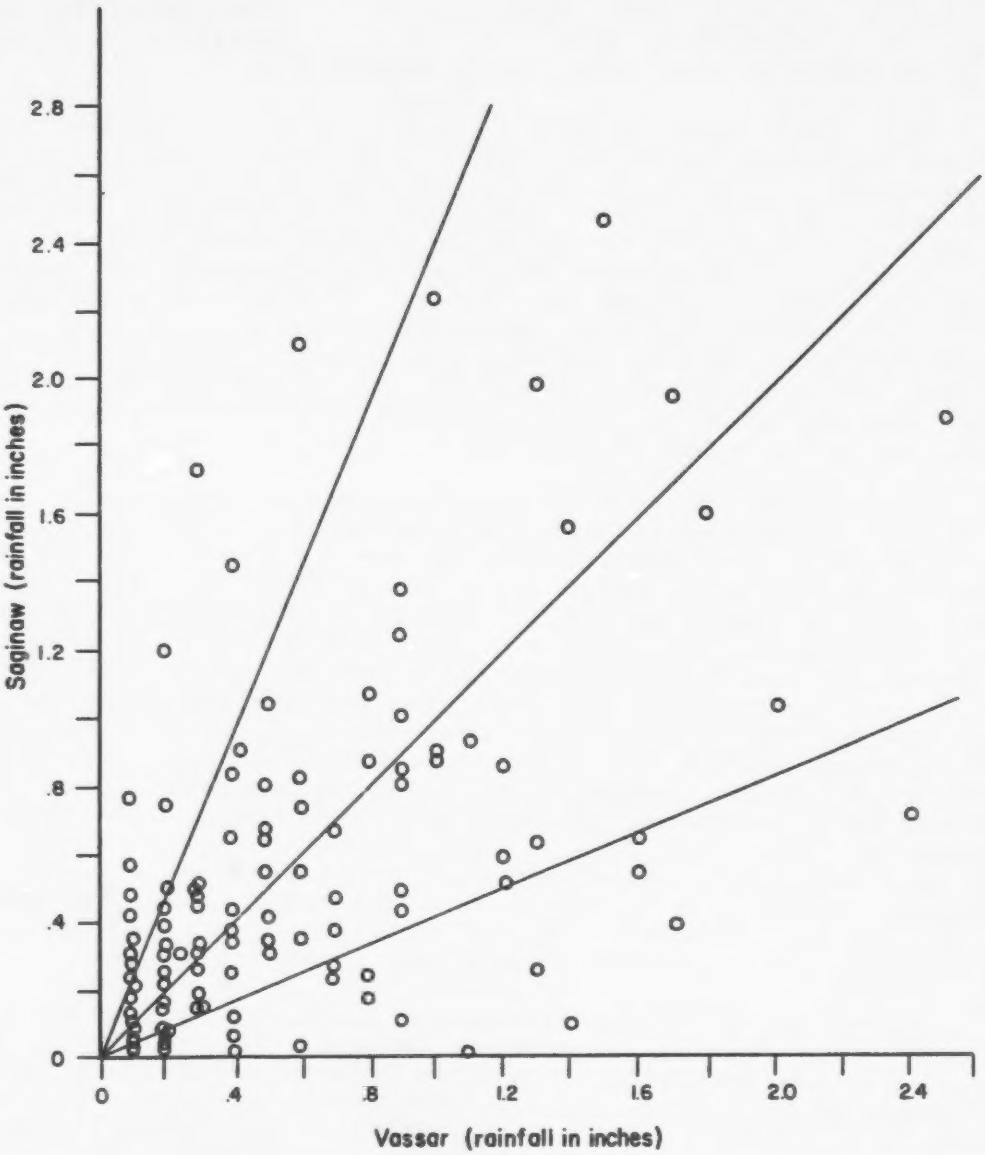


Figure 4.4 Comparison of storm events - Saginaw vs. Vassar - 1975 to 1978.

TABLE 4.4

COMPARISON OF ANNUAL RAINFALL TOTALS

ANNUAL RAINFALL (INCHES)

YEAR*	Vassar	Sebewaing	Tri-City Airport
1956 - 1965	N.A.	25.35**	N.A.
1969	22.12	23.09	20.14
1970	34.41	26.61	N.A.
1971	22.09*	17.54***	N.A.
1972	34.44	24.06	N.A.
1973	28.58	25.11	N.A.
1974	28.00	25.90	N.A
1975	36.00	25.92	N.A.
1976	35.50	22.18	30.05
1977	33.00	26.20	29.33
1978	21.00+	16.28+	20.97+

*	Calendar year
**	Computed, average per year
***	Missing November
+	Missing December

N.A. - not available

Annual rainfall totals given in Table 4.4 suggest that Sebewaing (for a different ten year period) had received less total rainfall than Saginaw and Vassar during the ten year period analyzed in this report. A frequency plot comparison shown in Figure 4.5 of Vassar rainfall (1969-1978) to the plots prepared for Sebewaing (1956-1965) in the previous study (1) and for Sebewaing (1969 - 1977) in this study verify the differences in rainfall exhibited by the annual results given in Table 4.4.

It is interesting to note that the one-year design storm of 1.9 inches used in the design of the Hancock Street facility derived from an analysis of Sebewaing data (1956-1965). The one-year design storm for the next ten year period at Sebewaing equals 1.45 inches. The rainfall plot at Vassar indicated that the storm with a recurrence interval of one year has a cumulative volume of approximately two inches. Using a strict definition of 6 dry hours between events, an average of 67 events per year was calculated for the Vassar Station, with an average of 0 .3 inches per event. The average antecedent dry period between rain events at Vassar was estimated to be four days.

A full year rainfall period was used in the simulation modeling efforts. A full year of overflow events would provide a better assessment of overall CSO control efficiencies since a variety of different storm intensities including low intensity but high frequency events would be included. The year of 1977 at Vassar was selected for the model simulation runs. The annual total rainfall was slightly above average. It had numerous small events, yet also contained several events in excess of the one year storm volume of 1.9 inches used in the prior study, and the one-year recurrence storm for the Vassar Station. The frequency plot for 1977 is also shown in Figure 4.5.

4.7.2 Catchment Areas

Catchment areas were outlined for each of 31 regulators but one minor area was eliminated and was added to a larger tributary area (there are a total of 34 regulators) while two other areas have alternative overflow points depending on river conditions and potential combined sewer overflow points. These locations and associated catchment areas are summarized in schematic form in Table 4.5. The areas were outlined on the City sewer map. Divisions between catchment areas were determined primarily by review of the sewer system plans. EDP inspected a number of points along catchment boundaries for further clarity. These catchment areas are roughly depicted on Figure 4.2.

It was noted in delineating the catchment areas that the older sewerage system had been upgraded by a number of relief

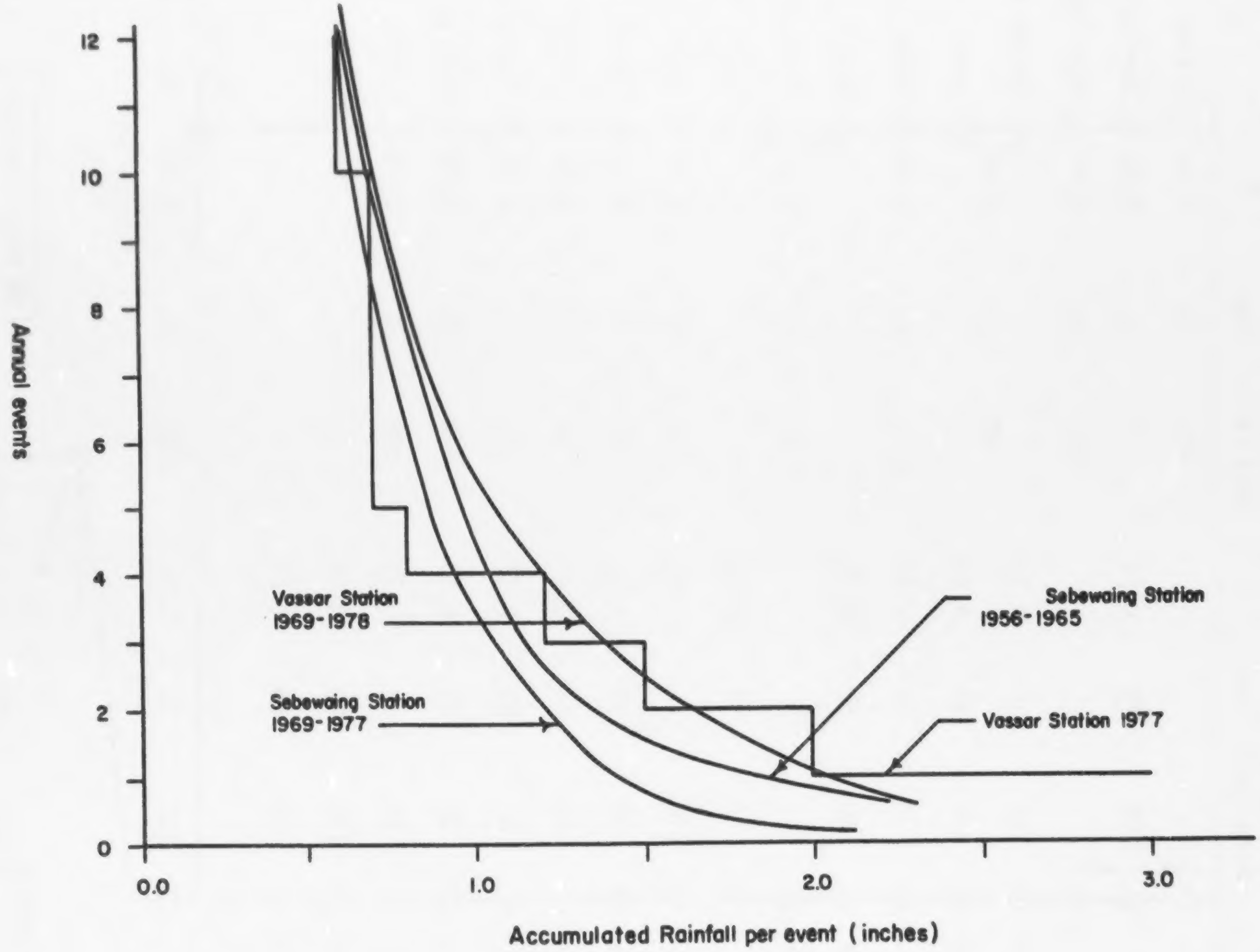
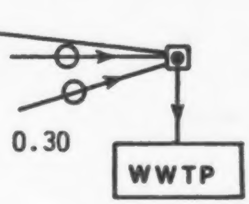


Figure 4.5 Annual rainfall (accumulated) cumulative frequency plot.

TABLE 4.5 CATCHMENT AREA /RUNOFF COEFFICIENTS

		Catchment Area (ac)	% Imp.	Z	Catchment Area (ac)	% Imp.	Z		
Webber	○	1020	25	0.34	440	35	0.41	○	Fraser
Webber	▣				10	60	0.60	○	Dearborn
Birch	○				470	60	0.60	○	Mackinaw
Holland	○	150	10	0.23	45	95	0.86	○	Van Buren
McCoskry	○	550	40	0.45				○	Cass
Holden	○	70	70	0.68	1030	40	0.45	○	Adams
Emerson	○	30	85	0.79	45	95	0.86	○	Court
Emerson	▣							▣	Hancock
Hoyt	○	43	80	0.75				○	Ames
Thompson	○	290	35	0.41	160	50	0.53	○	Hayes
Millard	○	30	90	0.83	240	90	0.83	○	Throop
Janes	○	90	40	0.45	20	90	0.83	○	Miller
Federal	○	80	50	0.53	170	70	0.68	○	Remington
Genesee	○	36	90	0.83	470	30	0.38	○	Genesee
Johnson	○	60	80	0.75	380	30	0.38	○	Cronk
Fitzhugh	○	130	25	0.34					
Carlisle	○	90	30	0.38					
								○	Weiss
First	○	190	15	0.26	2100	20	0.30	▣	Weiss
14th Street	○	1020	20	0.30					
16th Street	○								



▣ Pump Stations
○ Regulators

Z = Runoff coefficient

systems on both sides of the river meant to provide additional capacity during storm events. The result was a complex network of pipes and overlapping stormwater catchment areas.

A number of bifurcation points where new relief sewers connect into older systems caused uncertainty in how flow divides at these points. An EDP field inspection program noted dry weather flow directions and also checked diversion weirs not noted on the sewer plans. A hydraulic analysis was performed to estimate the relative flows in different directions at division points during high flow events. Dynamics were considered if obvious, otherwise flow was apportioned to the relevant catchment areas using a Manning's equation formulation. Fixed flow factors were computed assuming half-full flow conditions in the upstream pipe and the plan and profile slopes of the two downstream receiving conduits. This approach was deemed suitable for the purposes of dividing average hourly discharge in the simulation model.

Bifurcation estimates were prepared for the West Side intersection points at Brockway and Thurman, Adams and Congress, Houghton and Clinton, State and Carolina, Union and Woodbridge, State and Woodbridge, Superior and Vermont, W. Remington and Harrison, State and McEwen; and on the East Side at Remington and Alger, Holland and Good, Hoyt and Sheridan, Weadock and Johnson, and Seventh and Johnson. The results were used to divide the shared drainage areas between the competing regulators.

There were also a few separated sewer areas not included in the network, particularly the area south of Webber Street and the drainage system for the interstate highway. Dry weather flow from all separated areas were included in the modeling efforts. Runoff from all separated areas draining directly to the river was not included in the modeling effort.

4.7.3 Runoff Coefficients

Estimates of imperviousness and calculated runoff coefficients for each of the catchment areas are summarized in Table 4.5. Estimates of imperviousness were prepared from aerial photographs of Saginaw. Representative sections in a few of the major drainage areas were analyzed by comparing the areas occupied by housing and streets to open spaces, to arrive at estimates for percent imperviousness. These estimates were compared to standard percent imperviousness factors for different types of land use to check for reasonableness. The 31 different drainage areas were heuristically scaled to the major areas analyzed in detail, yielding percent imperviousness estimates for each catchment area in the system. The total system average was 34% imperviousness.

Actual runoff coefficients were then calculated on the basis of the percent imperviousness calculations. Standard procedures presented in simplified SWMM(2) were computed using equation 1:

$$Z = 0.15 + 0.75 \times I \qquad (1)$$

where Z is the runoff coefficient representing the fraction of rain that appears as runoff, and I is the percent imperviousness of the catchment area (in fractionalized form).

4.7.4 Stormwater Pumping Stations

Of particular interest to the storm overflow analysis are the five major pumping stations designed to pump flood waters and/or combined sewer overflows to the river. The Hancock Street Station on the West Side is presently connected to a 3.6 MG storage basin, with eight - 100 cfs pumps available. Further downstream of Hancock Street Station is the Weiss Street Station with three - 100 cfs pumps. On the East Side the Emerson Street Station has eight - 100 cfs pumps. The Emerson Street Station along with Hancock and Weiss are the only stations that can pump water directly from the interceptor with flows depending on the heights of weirs separating the interceptor from the pump station wet wells. The two other pump stations, Webber and Fourteenth Streets both have gravity outlets to the river, and only operate when river elevations are high enough to prevent the backwater gates at the gravity outlets from discharging the high runoffs. Both Webber and Fourteenth Street Stations have three - 100 cfs pumps each.

The amount of time each pump in the system operated was periodically recorded, and this information was used to analyze the operation of the five pumping stations. It was noted that the Weiss and Hancock Street Stations are frequently used with Webber and Fourteenth Street Stations operating much less frequently, and Emerson Street Station (manual start-up operation) only rarely . This comparison is not surprising considering the hydraulic bottleneck at the West Side river crossing, (see Chapter 6) the relatively high weir height at the Emerson Street Station above the East Side interceptor and the potential at the Webber and Fourteenth Street Stations for gravity discharges short-circuiting these pump stations.

Curves were developed by EDP for the Weiss Street and Hancock Street Stations relating total volume pumped to the amount of rainfall per event. Distinct rain events were associated with the amount pumped at these two overflow points by comparing pump usage records to rains recorded at the WWTP. It was noted, as expected, that both volume pumped and the number of pumps used increased with increasing amounts of rain. Spring events resulted in higher volumes pumped due to infiltration and spring melts. In addition to being a useful indication of system stress points, the pumping station analyses were used as a rough check of the simulaton model. Data plots of Weiss and Hancock Street Stations' pumping operations, as functions of total rainfall per event, are presented with the simulation model's predicted values in Chapter 6.

CHAPTER 5

SCREENING OF COMBINED SEWER CONTROLS

5.1 Foreword

The development of realistic CSO abatement concepts for Saginaw included determination of the degree of abatement required, assessment of available technologies and identification of realistic options based on that technology assessment. In this chapter available CSO alternatives ranging from inexpensive system modification to total capture and treatment of overflows are reviewed. The most promising and feasible abatement options are selected in this Chapter and are further described in Chapter 6.

The three general approaches listed below briefly describe the state of the art of CSO abatement:

(1) sewage system structural modifications to improve system operations and to enhance hydraulic conveyance capacity together with improved sewer system flow controllers at regulation points to enhance operational ease in flow splitting and/or to create maintenace free in-line transient detentive storage; (2) pollutant source control and upstream stormwater management practices, and; (3) combinations of major off-line storage and treatment facilities.

Existing system modifications are reviewed in section 5.2. Source control practices are reviewed in section 5.3. Storage and treatment options are discussed in section 5.4. Selection of feasible practical and realistic alternatives for further evaluation is discussed in section 5.5.

5.2 Modifications to Existing System

Reduction of overflow discharges can be achieved by a wide range of modifications from low cost, minimal structural procedures involving maintenance improvements to structural intensive sewer separation. The following methods were investigated:

Regulator Improvements - Utilization of new vortex valve technology to enhance the capability of existing regulators to efficiently pass "first flush" to WWTP in a relatively maintenance-free fashion and to effectively control underflows to the interceptor systems at maximum storm conditions. Transient in-line system storage configurations can be developed where applicable, at the regulating chambers by modification of chamber´s internal weirs and inclusion of positive discharge control devices. A discussion of vortex valve flow throttling equipment is presented in section 5.2.1.

Sewer Separation - The separation of storm and sanitary sewers significantly reduces combined sewer overflows. Sanitary flows are treated prior to discharge to receiving waters. Storm runoff pollutants discharge directly to receiving waters unless additional treatment is provided. To achieve total separation, the difficult task of disconnecting roof leaders is necessary. The principal disadvantage of sewer separation is its tremendous cost and time requirement.

Improved Utilization of Existing Facilities - Unused and/or increased capacity for conveyance/storage within existing sewerage system can often be realized at little cost. Increasing and/or improving hydraulic capacity at critical control points, i.e., pumping stations, constrictions, etc., are classical examples.

In-line Storage - System control using in-line storage represent promising alternatives in areas where conduits are large, deep, and flat (i.e., backwater impoundments become feasible) and interceptor capacity is high. Costs for storage capacity gained in this manner range from 10 to 50 percent of the cost of off-line facilities. Because system controls are directed toward maximum utilization of existing facilities, they rank among the first of alternatives to be considered.

5.2.1 Vortex Valve Throttling Devices
Existing State-of-the-Art Regulating Devices

In general, the purpose of flow regulating devices is to control discharge from a combined sewer into an intercepting pipe leading to the WWTP. In practice, three classes of devices are used: static, dynamic (semi-automatic), and dymanic (fully-automatic)(1).

The schematic of a dynamic (semi-automatic) device used in regulator chambers in Saginaw is depicted in Figure 5.1. During normal dry weather flow conditions, the float is in the down position permitting maximal aperature opening on the orifice discharge leading to the interceptor. During a wet weather event, the flow level within the combined sewer rises and flow enters through the telltail opening into the float chamber. As

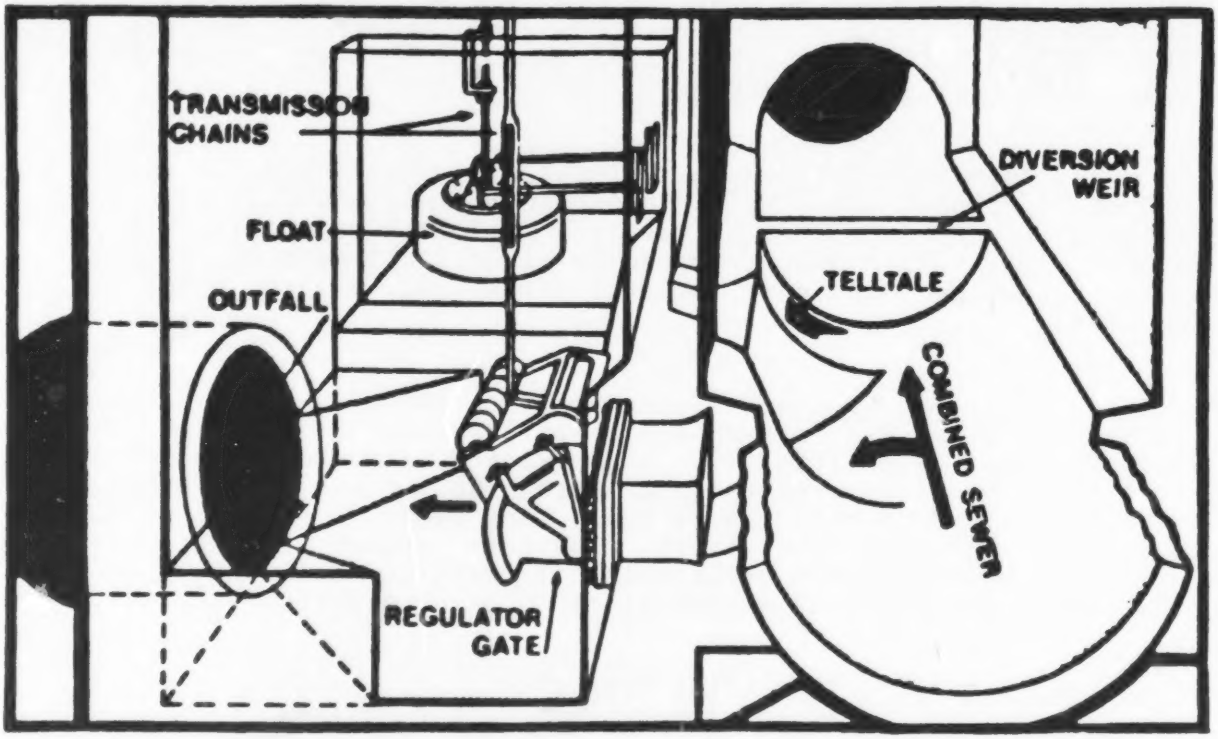

TRANSMISSION CHAINS
FLOAT
OUTFALL
REGULATOR GATE
DIVERSION WEIR
TELLTALE
COMBINED SEWER

Figure 5.1 Schematic of Typical Dynamic(Semi-Automatic) Combined Sewer Mechanical Float-operated Flow Controller

the level of water in the chamber increases, the float rises, and through mechanical connections, the moveable shutter lowers, thus continually decreasing the orifice aperature. As the orifice aperature decreases with increasing head during a storm event, the resulting flow to the interceptor will be throttled to predetermined limits. Mechanical float-operated systems of this type were state-of-the-art technology in the 40's and 50's and thousands were installed throughout the U.S. and Canada.

Regulation is controlled by movements of the float. In the larger sizes, the float diameter may be as much as 5 feet. This requires a large size floatwell which may trap grit that creates a maintenance problem. Accumulation of floating material on the float may cause malfunctioning of the system. Since the entire system is in fine balance, proper operation requires at least biweekly maintenance (1). Without proper maintenance the shutter can "stick" and "freeze-up" resulting in an unknown aperature opening and hence, unknown discharge. Frequently the shutter head "sticks" in a down position after a storm event due to its inertial and must be released by physically "kicking" or jarrying the head. An additional problem arises when the devices must operate on high head differentials. As the shutter opening decreases, the risk of cloggage increases. Operational problems with this type of device needs no documentation.

Justification - Vortex Valve Technology

Over the last several decades significant improvements in sewerage system control have occurred in the USA, Germany and Scandinavia. It has been recognized that a further improvement of the efficiency of WWTP will, when viewed in overall terms, have relatively little effect as long as a large proportion of the pollutants fails to reach the WWTP during wet weather. This applies particularly to combined sewerage schemes with numerous storm overflows.

An improvement of the efficiency of the collecting system is possible, by installing additional reservoirs with heavily throttled outflows in the the form of stormwater detention reservoirs, storm water overflow detention/retention tanks or through the use of in-system transient storage configurations. This type of control strategy, however, makes discharge control from such collecting system storage configuration, a new and central problem.

Modern CSO control philosophies focus on the "first-flush" to separate the dirtier water from the less polluted. This can be done easily and reliably the shorter the combined passages of foul water and storm water outflows are. This means that collection system storage configurations will have to be present in greater numbers and will have to be decentrally situated. It follows then that these units may be infrequently serviced, but nevertheless, they must operate reliably.

The sophisticated methods of urban hydrology employing mathematical models to accurately estimate required storage volumes lose this effectiveness if the flow control element fails to maintain the desired flow. The accuracy and reliability of conventional type throttles as described above are not reliable nor are they accurate (2). Table 5.1 summarizes in a general sense characteristics encountered and design parameters for an ideal remote type sewerage system flow controller. The vortex valve technology satisfies these requirements.

Historical Background of Vortex Valve Technology

The roots of vortex valve technology date back many years. Around 1930 a non-return valve or vortex diode was invented in Germany in order to reduce the danger of uncontrolled blow-off of hot steam in the case of pipe fractures. This invention was left without any practical application until the sixties. It was then re-discovered and was modified and used for control functions in which maximum reliability was required, such as the control of rocket motors and emergency cooling circuits for nuclear power stations. In Germany so-called low-pressure vortex amplifiers have been developed since about 1970 for purposes of hydraulic engineering (3). The first practical applications began in 1976 (4). In Germany there are about 1000 vortex devices of different types and size in operation. A patent was applied for by H. Brombach in USA and was received.

At the same time, independent experiments were conducted in Scandinavia with vertically arranged "Hydro-Brakes." These were tested as inlet throttles in catch basins. Later conical in-line types were developed. Hydro-Brakes have their origin in this branch of development. There are several thousand such devices in operation in North America, mostly the catchbasin throttle type. A patent was applied for in USA by J. M. Johannessen and was received.

Simplified Justification of Vortex Flow Controllers

Vortex devices work exclusively with flow effects. There are no moving parts and there is no external energy supply. The flow effects are three-dimensional and determined by fluid friction. To date, there is no satisfactory quantitative

TABLE 5.1

Conditions and Demands for an Ideal Collecting System Flow Controller (2)

Conditions:	Requirements:
decentral and multiple installation	simple and easy to understand function
rare supervision, installed in wet and dark	simple maintanence, no special tools or meters required
reliability	no moving parts, no external energy supply, self-cleaning
small flows	large cross sectional openings, no backwater in dry weather
long-term operation	durability, corrosion protection
refined hydrology	accuracy, stable characteristics
changes in catchment	adjustable flow rates

description of the flow processes by means of a mathematical model.

The operating principles of this technology can be demonstrated considering the simplified model shown in Figure 5.2. An infinitely large container is connected via a channel (C) to a cylindrical vessel (V). The vessel (V) has a centralized hole at the bottom of diameter (Do) or of area (Ao). If the pipe (C) is routed so that the water enters the vessel (V) perfectly radially, e.g. via a ring pipe with small radial nozzles, (see Figure 5.2A), the result is a sink flow in the direction of the outlet orifice. The outflow jet is constricted so that the flow rate becomes

$$Q_r = A_o\ m_r \sqrt{2gh}$$

If the outlet is of sharp-edged design (orifice plate), the loss coefficient is approximately, m_r =.6.

Figure 5.2B shows the same setup, but the inlet nozzles are arranged tangentially. The difference is striking. There is a free vortex generated under these conditions. The nearer a water particle comes to the center line of the vessel (V), the greater its peripheral velocity becomes. The centrifugal force becomes so great that a vortex core is formed through the outlet hole and allows the water to flow out in the form of a hollow jet. This effect can be observed in a bath tub vortex. The flow obeys the same function

$$Q_t = A_o\ m_t \sqrt{2gh}$$

but the flow coefficient drops to approximately, m_t =0.15.

A particularly remarkable feature of this vortex flow is that each disturbance of the flow, e.g. due to entrained solids or air pockets, leads to a weakening of the centrifugal force and thus to an increase of the flow. This unusual hydrualic behavior. also known as the self-cleaning effect. has obvious advantages in flow control of waste streams with solids and debris.

Although the two experiments in Figure 5.2 differ only slightly in the arrangement of the inlet nozzles, the flow is very different. namely

$$Q_t/Q_r = 1/4$$

PHYSICAL JUSTIFICATION FOR VORTEX VALVES

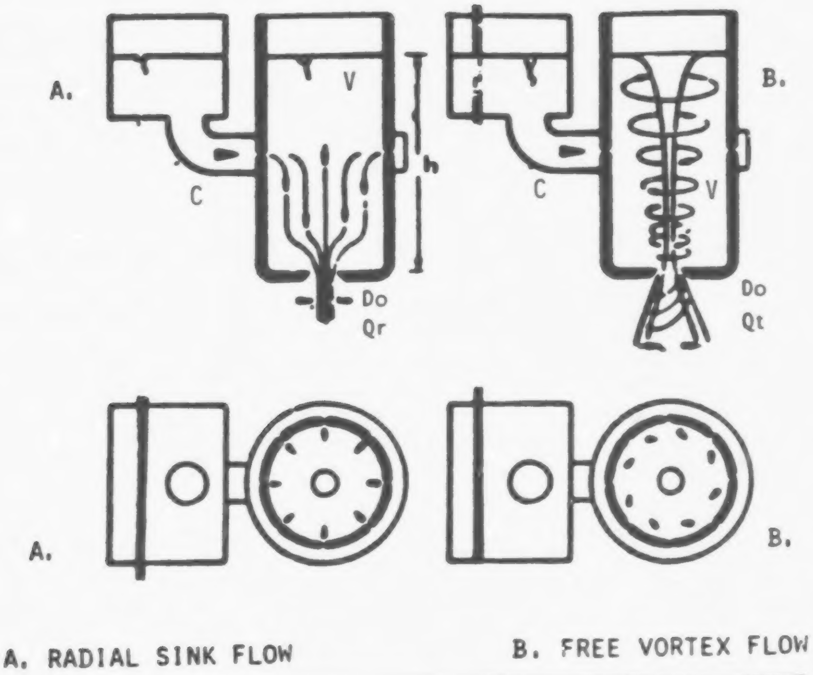

A.
V
h
C
Do
Qr
B.
C
V
Do
Qt
A.
B.
A. RADIAL SINK FLOW
B. FREE VORTEX FLOW

Figure 5.2 Simple Model of Vortex Valve Representation

The flow rate is reduced to about 1/4 as a result of the tangential supply of water. Expressed differently, it is possible under vortex flow conditions to obtain the same flow as a simple orifice, but the area (Ao) of the outlet nozzle will be 4 times the size of the comparable simple orifice. If it were possible to switch automatically between the two flow states, the result would be flow valves whose flow rate is adjustable in the ratio of 1:4.

Vortex flow controllers can be designed to uniquely combine both types of orifice and vortex flow principles into a single stage/discharge characteristic. Operation descriptions of how this combination of flow effects occurs are provided below for vortex valve equipment of Scandinavian origin and then for vortex valve equipment developed in Germany which are relevent for combined sewer regulator applications.

Conical Type Vortex Valves - Scandinavian Origin

In 1980 J.M. Johannssen of Denmark tested and developed an in-line vortex control device for in-line applications (inlet and outlet invert elevations are the same). The conical device shown in Figure 5.3A can be described as follows. It includes a truncated cone section, a circular plate at the inlet end, a cylindrical inlet, an oval plate at the outlet end and a cylindrical outlet.

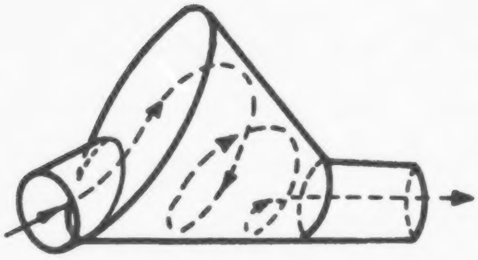
The cylindrical inlet section consists of a formed circular section which lies in a level horizontal position along its longitudinal axis and is diagonally cut such that the inlet intersects the cone body at a 45 degree angle. The inlet plate is circular and is solidly welded to the large end of the truncated cone and includes an oval orifice located at the bottom of the plate. The orifice exactly matches the diagonally cut surface of the inlet segment. The inlet segment is solidly welded to the inlet plate at the inlet plate orifice. The truncated cone section acts as the main body of the device and provides structural rigidity. An oval-shaped end plate is solidly welded to the small end of the truncated cone segment. Integral to the end plate is a circular orifice at the base of the oval end plate sized to exactly match the truncated cone small end. In the installed position the oval end plate is parallel to the circular end plate on the inlet end. The cylindrical outlet is a roll formed pipe solidly welded to the oval end plate, and is diagonally cut at the same angle as the truncated cone such that the outlet tube exactly matches the oval end plate. The oval end plate and cylindrical outlet are sized as necessary to match field conditions.

Cylindrical outlet segments normally incorporate rubber "O" ring stops for standard push-fit installation. Slip flange ends, victaulic coupling ends, clamps or other specialized end connections are constructed as required for special connections.

J. M. Johannessen had earlier developed in 1977 a proto-type conical device with an inlet slot cut along the longnitudal axis of a truncated cone with an internal orifice located at the outlet back plate. Incoming flow would impact the back plate and then enter the device through the inlet slot. A family of stage discharge curves is generated by altering the internal orifice dimension. The large end of the truncated cone is a fixed multiple of the internal orifice. He was not satisfied with the performance of this prototype unit for the following reasons: a) the flow-through pattern was not satisfactory; b) the vortex braking effect was not satisfactory; c) the rated flow capacity of the unit was dependent on the mounting position of the inlet slot (any deviation from absolute vertical alignment would alter the flow characteristic; and, d) the structural rigidity of the cone was greatly reduced due to the inlet slot along the cone length (typical hydraulic surges, i.e., vibrations may cause large units to collapse) (5). It was for these reasons that he developed the conical type shown above in Figure 5.3.A.

A typical stage/discharge characteristic for the conical device developed by J.M. Johannessen is shown in Figure 5.3.B and is described as follows: As the unit begins to fill under increasing head, it will discharge similar to a sharp-edged orifice (see segment 0-1, Figure 5.3.B) With greater head, air will become trapped in the upper portion of the chamber and a special characteristic will be formed as the discharge increases along segments 1-2-3. Finally, when the head level increases above the chamber, full braking effect is achieved along segment 3-4, (Figure 5.3.B). As the need for braking decreases (falling head), discharge follows sements 4-3-5, (Figure 5.3.B and then drains as a sharp-edged orifice. This sudden increase in discharge provides a desireable flushing effect on the unit as to maintain self-cleaning conditions.

Families of stage/discharge curves for this conical configuration can be generated by simultaneously altering the dimensions of the truncated cone large end and the internal orifice located at the truncated cone small end. For example, increasing the dimensions of the truncated cone large end while keeping the internal orifice fixed increases the vortex braking effect. Thus, Johannessen greatly increased design flexibility with the improved conical version. The opening of the inlet section is either equal to or greater than the internal orifice. The outlet discharge pipe is always equal to or larger than the



A. **AFI MC Series**
Inlet and outlet at same elevation

B.

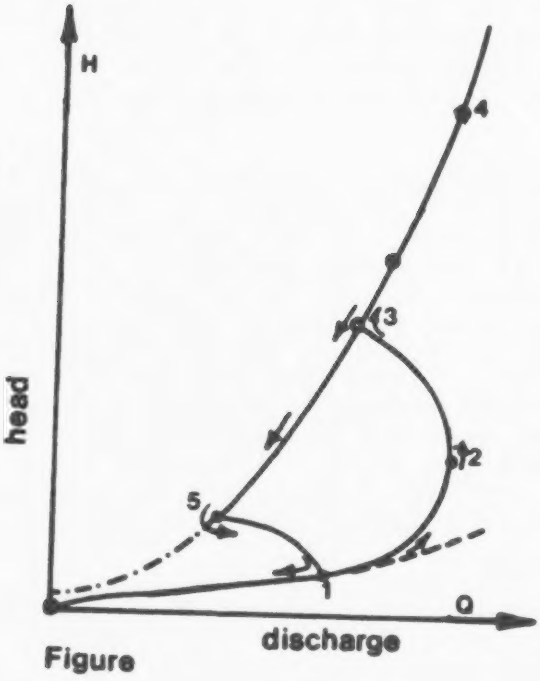


Figure 5.3 Johannessen Type Conical Vortex Valve Flow Controller

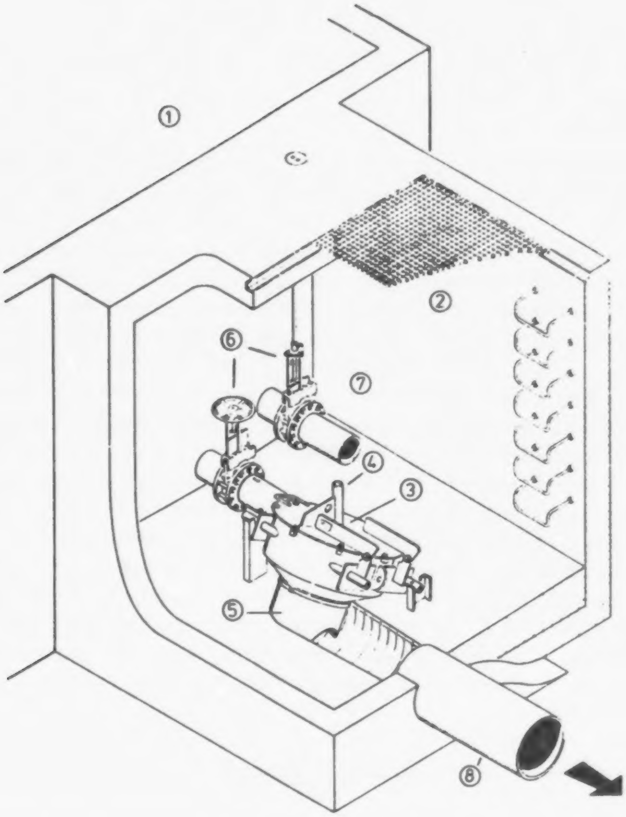
internal orifice. The commercial technology discharge range for the Johannessen conical vortex valve is 0.3 cfs to about 20 cfs.

The coefficient of discharge in the vortex braking mode for Johannessen´s device ranges from 0.18 up to 0.37, depending upon the particular geometry configuration. These coefficients can be favorably compared to the mechanical float-operated regulator shown in Figure 5.1, which vary from 0.95 (100% shutter opening) down to 0.725 (5% shutter opening)(6). It is for this reason that vortex valves can throttle discharge with much larger aperature openings in comparison to a standard orifice. The vortex valves can pass more efficiently debris and solids through the chamber due to the spiraling flow action created by the vortex action. Furthermore, the vortex valve can operate as an orifice under low head conditions (and pass large flows) and the switch to the vortex mode with increasing head. Vortex valves can automatically switch to an fro, from a orifice characteristic to a vortex mode. The notion of the name "vortex valve" derives from the device´s ability to vary flow resistence as a function of inlet pressure.

Concial Vortex Valves - German Origin

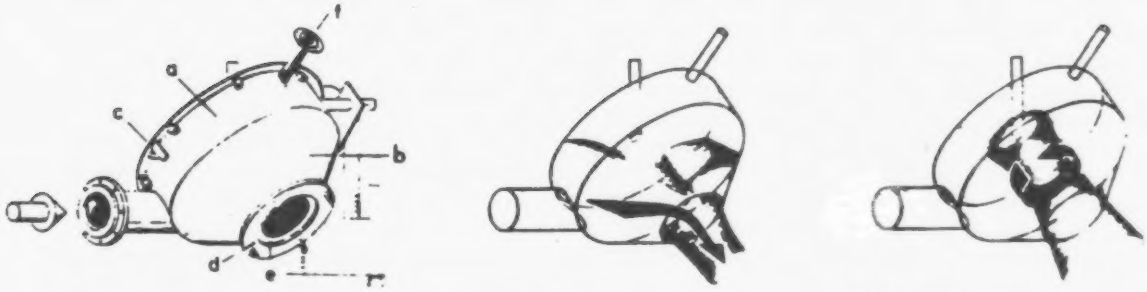
In the late 70´s Dr. H. Brombach, West Germany developed the conical vortex valve, shown in Figure 5.4.A. It consisted of an inclined housing. The lower generating line of the conical part of the housing is horizontal. A tangential inlet pipe joins into the housing. The size of the central exit port is variable by means of exchangeable orifices. The housing has a hinged cover as well as vents (an important stabilizing improvement).

The switching to and fro between radial sink flow and free vortex flow occurs as follows (2): If the pressure at the tangential inlet to the vortex chamber is low, then the housing is filled with water only in the lower part. Due to the included position of the housing the body of water is not rotationally symmetrical. Despite the tangential inlet it is not possible for any vortex flow to develop, but instead there is more of a nozzle flow of low flow resistance, see Figure 5.4.B. The higher the pressure in the inlet, the more completely the vortex chamber is filled with water and the excess air escapes. The body of water loses its non-rotational symmetry more and more. When the chamber is fill to the peak, Figure 5.4.C, the inclined position of the chamber no longer has any effect since water is weightless in water. The vortex now operates unhindered by asymmetry and greatly throttles the flow. The central air opening draws air into the vortex core. The vent at the top of the chamber is subjected to over pressure and must, therefore be extended beyond the highest water level.



① storm water tank
② control shaft
③ vortex flow controller
④ vent
⑤ splash protection
⑥ slide valves
⑦ bypass
⑧ pipe to sewage treatment plant

Typical installation of a conical single inlet vortex valve



Automatic Vortex Flow Controller

A.

low resistance

B.

high resistance

Flow pattern in the valve housing

C.

a valve housing
b cone
c hinged cover

d exit port
e dischauge ba
baffle
f vent.

Figure 5.4 Brombach Type Conical Vortex Valve Flow Controller

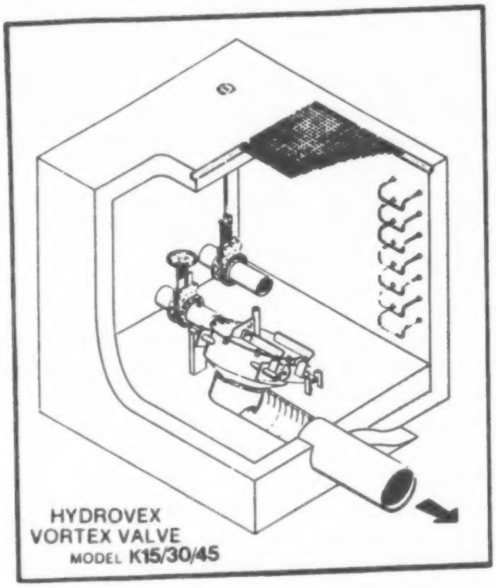
Figure 5.5.A shows the hydraulic behavior of a model of valve used in numerous applications in Germany. The left-hand part of the figure shows the water levels in the vortex chamber at various inlet pressures and flow rates. If the rate and flow coefficient based on a pressure of 15 inlet diameters are plotted, graphs shown in Figures 5.5.B and 5.5.C result (2). The exit orifice has the same cross sectional area as the tangential inlet.

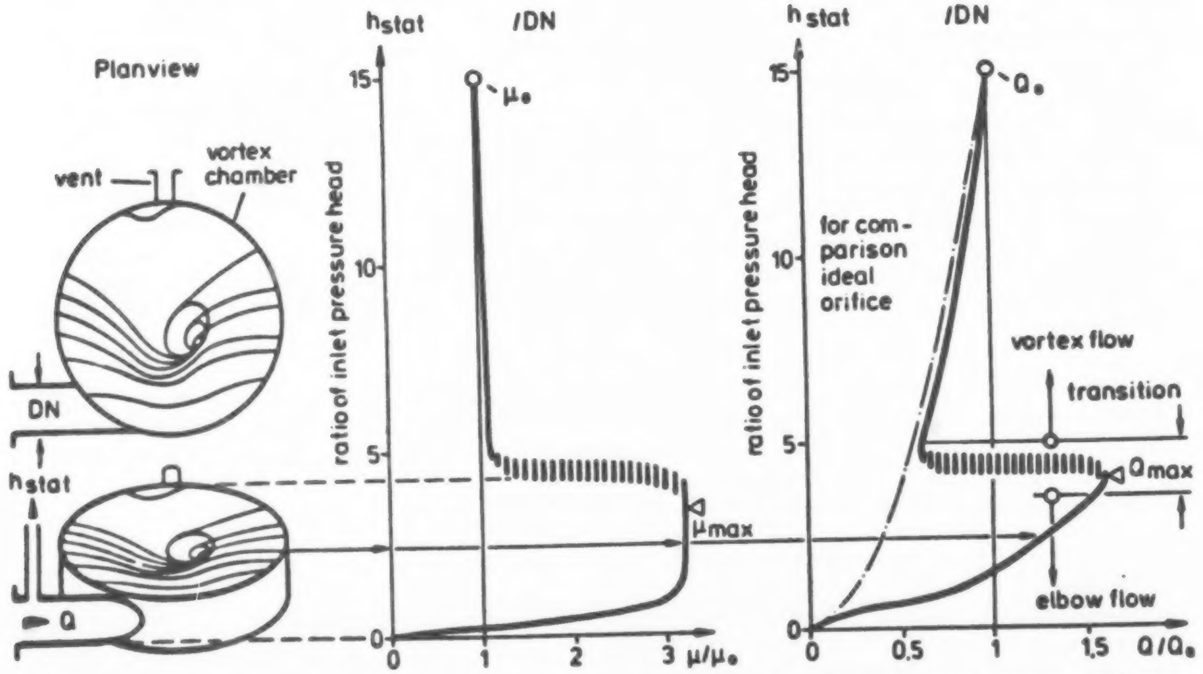
It can be seen by inspection of Figure 5.5 that there are four different flow characteristic regions. The lower-most section, partial filling, is the region in which the water runs through the device with a free level. The rapidly increasing flow coefficient (see Figure 5.5.B) is determined by the hydrualic behaviour of the tangential inlet. In the second section, the characteristic of the flow coefficient swings into the vertical and remains at a value of about 3.25. The maximum theoretical possible value of 4 is not quite reached. This is due to the fact that the incident flow on the exit orifice in this state is not entirely free of circulation. The complete filling of the housing is reached at a pressure head of about four inlet diameters. The flow coefficient drops sharply. In the third section, the transition zone, the device reacts very sensitively to fluctuations in pressure. This makes it difficult to obtain precise measurements of the flow coefficient. For this reason, this region is shown by a broken line. In the upper most zone of the characteristic, there is stable vortex flow. The water leaves the vortex chamber at high speed in the form of a swirling hollow jet. The flow coefficient drops slightly as the pressure rises. This is because the air-filled vortex core, which tends to rise upwards due to its buoyancy in the inclined chamber, is progressively driven towards the axis as the pressure rises and so is progressively less able to disturb the vortex action.

Figure 5.5.C shows the discharge-stage relation. The flow rate rises continuously to a peak flow and then drops back. This results in a saw-tooth-shaped flow curve. By comparison, the flow curve of a simple orifice is shown. This achieves the same desired flow at the end of the curve.

The characteristic of the German vortex valves is basically the product of the following four geometrical parameters: a) inlet diameter, b) outlet diameter, c) housing diameter, and, d) angle of inclination; and the pressure on the inlet side. If the valve does not freely discharge, but against a back-pressure, there is a further parameter.

Systematic variation of the geometrical parameters results in an entire family of vortex valves. Although the


HYDROVEX
VORTEX VALVE
MODEL K15/30/45



(A) Water levels at various inlet heads (B) Characteristics of flow coefficient (C) Characteristics of volume flow

Figure 5.5 Flow Coefficent and Discharge Variations For Brombach Type Conical Vortex Valve

hydraulic properties cannot be adjusted at will, there is a wide degree of flexibility. Thus, for example, there are valves with a highly pronounced saw-tooth, and others where the change from nozzle flow to vortex flow is such that the flow curve is almost vertical.

The hydraulic dimensioning of the W. German vortex valves is based on extensive full scale calibration measurements. In practical applications of the vortex valves, slide valves and connecting pipes are often positioned upstream of the valve itself. Correct dimensioning must, therefore, take account of the influence of these additional hydraulic resistances. Only in this way is it possible to guarantee an accuracy of +/-5% of the desired flow (7).

The discharge loss coefficients for the W. German vortex valves are below 0.15 and range up to 0.3. The commercial technology discharge range is 0.1 cfs up to about 100 cfs for a single device with no upper head limitation. In Europe there is an installation throttling about 1 cfs of stormwater containing sand and gravel under a head drop of 1600 feet. All of the W. German type vortex valves useful for combined sewer regulation are constructed with hinged hatches allowing for easy maintenance access and permitting rapid replacement of adjustable plastic and/or steel orifice inserts to alter the units stage/discharge characteristic. A wide range of different flow characteristic curves can be accomplished through replacement of inserts allowing for capacity expansion/reduction and seasonal effects.

In sum, the W. German devices developed by Dr. H. Brombach provide more degrees of design freedom than the Scandinavian devices and are based upon extensive full scale hydraulic testing. The W. Germany devices include venting systems which stablize flow characteristics during switching and minimize any hysterisis effects guaranteeing that rising/falling stage curves are nearly identical.

Combined Sewer Regulation Design Considerations

Figure 5.6 shows a typical vortex valve stage/discharge characteristic. As it can be seen from inspection, the lower portion of the curve increases (rising head) to a defined peak (orifice characteristic), "kick-back" occurs in the transition zone (energy re-arrangement from orifice to vortex made), and then rises when in the vortex mode. On falling head, the characteristic curve is essentially the same.

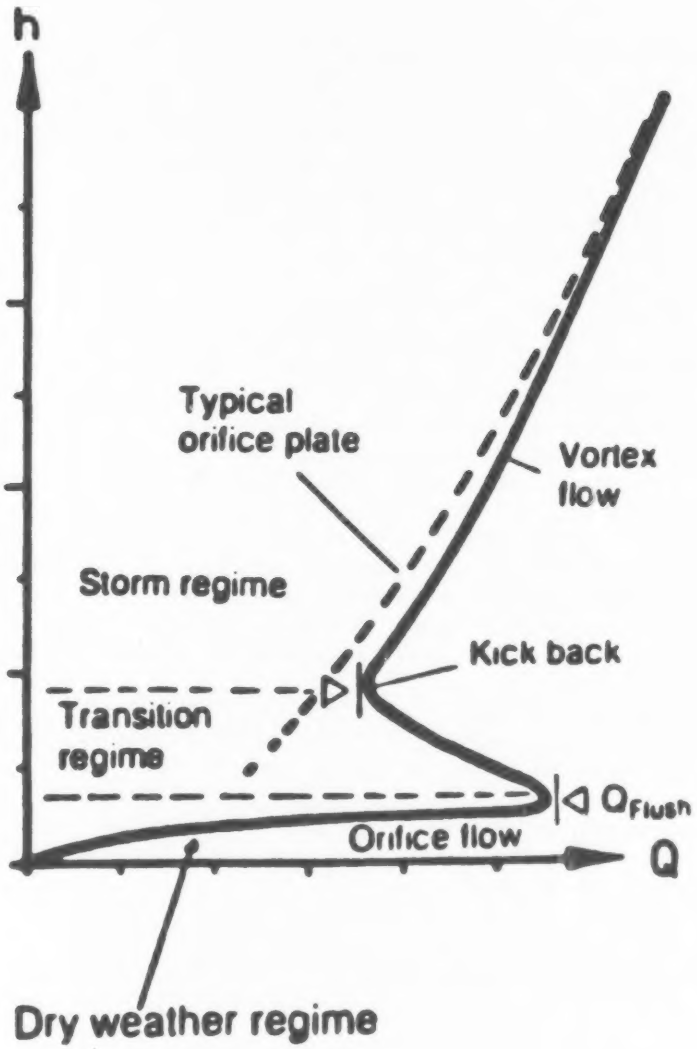


Figure 5.6 Typical Vortex Valve Stage/Discharge Curve

This characteristic provides for a number of interesting design possibilities. First, the "first-flush" passage can be designed about the initial peak. Second, overflow spill weirs can be accurately established when the vortex mode is first established. Third, the upper most portion of the vortex curve can be used to establish controlled flow rates at the maximum design storm condition i.e., 25 year design storm such that discharge at maximum head could be the same or even less than the peak "first-flush" flow occurring at low head conditions. The WWTP would therefore receive at most no more than either of these two hydraulic limits. Fourth, a system-wide utilization of these positive acting controllers could be employed to apportion interceptor capacity to carry only "first-flush" to the WWTP from each of the contributing sewer sheds and allowing only the clearer flows during increased storm conditions to overflow. Lastly, the orifice flow characteristic on drain-down (falling curve) can be explicitely used to ensure self-scouring when considering in-line and/or off-line storage of wastes heavily laden with settleable solids. It was exactly for this last design consideration that motivated Brombach to explicitly research, hydraulically test and develop vortex valves having well defined switching (orifice to vortex, vortex to orifice) characteristics.

The typical characteristic curve shown in Figure 5.6 can be altered by changes in the geometric design of a given vortex valve to increase/decrease peak initial flow; increase/decrease height at which the initial peak flow occurs; increase/decrease the degree of "kick-back"; and flatten/steepen the slope of the vortex mode portion of the curve.

Commercial Availability of Vortex Valves

Advanced Fluidics, Inc. (AFI) located in Hanover, MA acquired exclusive U.S.A. and Canadian licensing rights from J. M. Johannessen to market and sell his technology in 1982. These devices were sold under the trade name, Hydro-Brake. Conical type vortex valves were designed, fabricated and supplied to the City of Saginaw by AFI in mid 1984 and were installed as part of the reduced BMP implementation program.

John Meunier, Inc. (JMI) located in Montreal, Canada acquired North American licensing rights in-mid 1984 from Dr. H. Brombach to market and sell the patented line of West German vortex valves.

Recently, JMI and AFI have mutually agreed to merge the two vortex valve product lines and to combine licensing rights for marketing a a single product line called the "HYDROVEX" .

5.3 Source Control Practices

Management Practices include source control and collection system management. Whereas storage and treatment technologies are generally more costly alternatives, management practices concentrate on the source of pollutants (source control) and means of conveyance at a minimum cost.

A. Source Control technologies which were reviewed during screening are listed below:

Street Cleaning - Urban runoff pollution is reduced and aesthetics are improved by regular street cleaning. Fine solids fraction of street surface dirt is removed by broom brushes or the more efficient vacuum-type cleaners. The efficiency of street cleaning in large metropolitan areas is reduced due to problems with heavy street traffic and parked vehicles.

Solid Waste Management - Urban runoff pollution is mitigated by improving trash collection and reducing litter. The greatest benefit is aesthetic improvement rather than overall solids reduction.

Erosion Control - Improved construction practices, such as proper landscaping, protection of drainage channels, protection of stockpiles, and the installation of sediment retention basins/dams provide for a reduced fine solids contribution to existing sewer facilites.

Catch Basin Cleaning - There are several methods available for catch basin cleaning which will reduce clogging of storm and combined sewers. Removing a large fraction of the collected solids can reduce the solids pollution and prevent shock loadings of grit and silt materials.

Roof Storage - Additional storage of urban runoff for flow attenuation is available in an urban area of high building density provided that the roof structure is constructed to accomodate rainfall storage. A large amount of roof area is required to make this option attractive.

Parking Lot Storage - Similar to roof top garage, temporary storage may be available at parking lots and playgrounds. Outlet controls are necessary for this method to be effective. A regular maintenance program for debris removal of deposited material is necessary.

Natural Flow Attenuation Techniques - Natural drainage impoundment can be used for runoff attenuation by creating temporary detention. Specific examples are lagoons, debris dams, and recharge berms. These applications are not feasible for CSO problems in major urban areas.

Porous Pavements - The use of special pervious subbase and base course material for streets, sidewalks, and parking lots allow rainfall to permeate through the pavement for groundwater recharge. This technique is generally limited to new construction in mild climate.

B. Collection System Management Techniques - which were screened are listed below:

Polymer injection - The use of polymer can reduce hydraulic friction and thereby increase system conveyance capacity, but reasonable results can only be attained in hydraulic systems with limited capacity. Polymer injection is a temporary means of overflow control and is relatively expensive due to high chemical cost.

Sewer Flushing - High flows push settled solids downstream in sewer reaches which are too flat to be self-cleansing. Sewer flushing is usually limited to relatively small diameter sewers.

Sewer Cleaning - Periodic cleaning can restore sewer pipes to full hydraulic capacity using rodding machines, bucket machines and drag lines. These methods have proved to be relatively costly and time consuming.

5.4 Storage and Treatment

The use of storage and treatment technologies to abate CSO pollution encompasses a broad spectrum of possibilities ranging from structurally intensive, high cost alternatives to minimal structurally intensive, low cost alternatives. Structurally intensive projects are outlined in the following manner: System conveyance, storage and treatment.

System Conveyance - The collection and transfer of combined sewer overflow to treatment and storage facilities may be accomplished either by gravity or by pumping, depending upon the hydraulic conditions in the system. If storage is not available or is impractical, conduits and pumps must be sized to handle peak flows resulting from design storm runoff. With storage capabilities, conduits and pumps can be sized such that a combination of storage, conveyance, and treatment will process peak runoff at minimum cost. The CSO conveyance system may be continuous, collecting all overflows or selective, intercepting and combining only particular overflows. Hydraulic conditions, design storm, solids composition, and service area determine the size and shape of the conveyance facilities.

Storage - Off-line storage is used to attenuate storm flow peaks, reduce storm overflows, and capture the first flush, or provide treatment in the form of sedimentation when storage capacity is exceeded. Off-line storage facilities may be located at overflow points or near WWTP facilities, depending on the type and function of the storage facility to be used. Off-line storage may also be used for on-site storage of runoff. Disadvantages of storage facilities include their large size, high cost, and dependency on other treatment facilities for processing the retained water and settled solids. CSO generally results in solids handling problems so consideration must be given to conveyance system slopes and flows and also to the number of storage vessels which can be feasibly maintained. A typical process diagram is shown in Figure 5.7.

Treatment - Treatment facilities may be localized or centralized. The number and location of treatment facilities depends upon system hydraulics, characteristics of the pollutants and feasibility of operation. If hydraulic conditions are such that expensive pumping facilities are required for transporting relatively small amounts of overflow, then localized treatment may be more cost-effective. If hydraulic conditions allow consolidation of overflows, larger more centralized CSO treatment facilities become more feasible. The nature of pollutant loadings also determines the type and location of facilities. For example, if a relatively small area contributes a very high grit loading which would aggravate the conveyance system hydraulics, then it would be advantageous to locally remove the grit. Treatment technologies available for intercepted wet weather flows include physical alternatives which remove settleable, suspended solids and floatable materials from combined sewer overflow.

5.4.1 Physical Treatment Processes

Physical treatment systems have demonstrated capabilities to handle high and variable influent concentrations and flow rates and operate independently of other treatment processes except for the disposal of residual solids. Primary level treatment can be achieved by any of several methods available for high rate application, including gravity sedimentation, swirl regulator/concentrators, screening, dissolved air flotation, high rate filtration and high gradient magnetic separation. Descriptions of each of these methods are presented below.

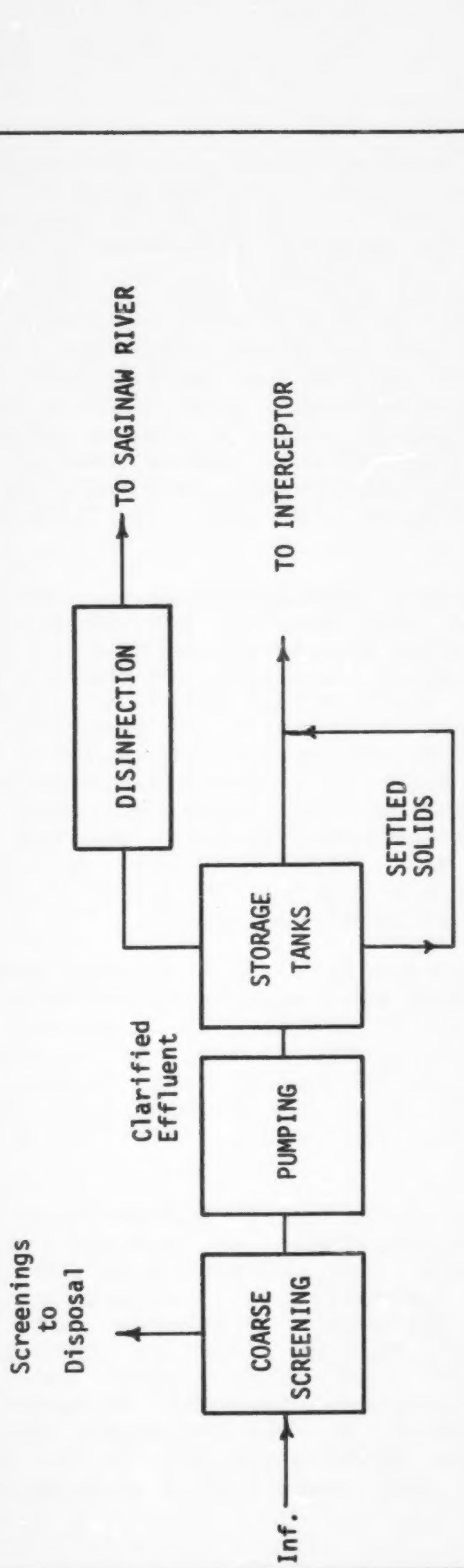

Inf.
COARSE SCREENING
Screenings to Disposal
PUMPING
Clarified Effluent
STORAGE TANKS
DISINFECTION
TO SAGINAW RIVER
SETTLED SOLIDS
TO INTERCEPTOR

Figure 5.7 Saginaw study typical storage facility process schematic.

Gravity Sedimentation - Gravity sedimentation of CSO has been evaluated in facilities utilizing surface overflow rates higher than conventional primary rates of 800-1,200 gpd/sq.ft. Early experiments with primary treatment facilities in Los Angeles demonstrated that suspended solids removal efficiences did not decrease markedly when overflow rate was increased over 2,000 gpd/sq.ft. Studies conducted on primary facilities treating combined sewage in Toronto, Canada; Rochester, New York; and New York City also show reasonable performance with increased overflow rates. Studies also show considerable cost/performance improvements with chemical addition such as polymer and alum. Despite the apparent improvements, gravity sedimentation requires tanks, thus introducing significant land requirements and solids handling problems similar to that of conventional storage facilities.

Swirl Regulator/Concentrators - Swirl regulator/concentrators achieve both quantity and quality control of CSO laden with suspended material. A typical process diagram is shown in Figure 5.8. The swirl functions on the basis of having a tangentially induced waste stream which is directed around the circular unit to form a spiral-type flowpath. Solids/liquid separation is achieved by two means: centrifugal forces separate and concentrate solids due to inertial differences; and solids fall out of the wastestream and are concentrated in the floor of the swirl due to gravitational effects derived from lowering particle velocities and elongating the particles flowpath. Floatables are effectively trapped and stored during storm event and later discharged to receiving sewer.

Extensive mathematical and hydraulic modeling has been performed with materials simulating particle size distributions and specific gravities encountered in combined sewage. These models were extended to prototype scale using Froude Law scaling of particle settling velocities and design flow rates. Verification of model projections has been demonstrated in facilities at Lancaster, PA; and at Rochester and Syracuse, New York.

A 12-foot diameter swirl concentrator in Syracuse, New York with a design flow of 6.8 mgd exhibited total mass loading removals for suspended solids ranging from 44% to 65% with concentration reductions ranging from 18% to 55%. The foul sewer discharge was approximately 10%. The higher mass loading reduction through the swirl concentrator is due to both flow and concentration reduction. Data performance from the Syracuse prototype unit indicate good removals at the beginning of storms (when influent concentrations were high) and at the end of the storm (when flow rates were low) such that a high percentage of the

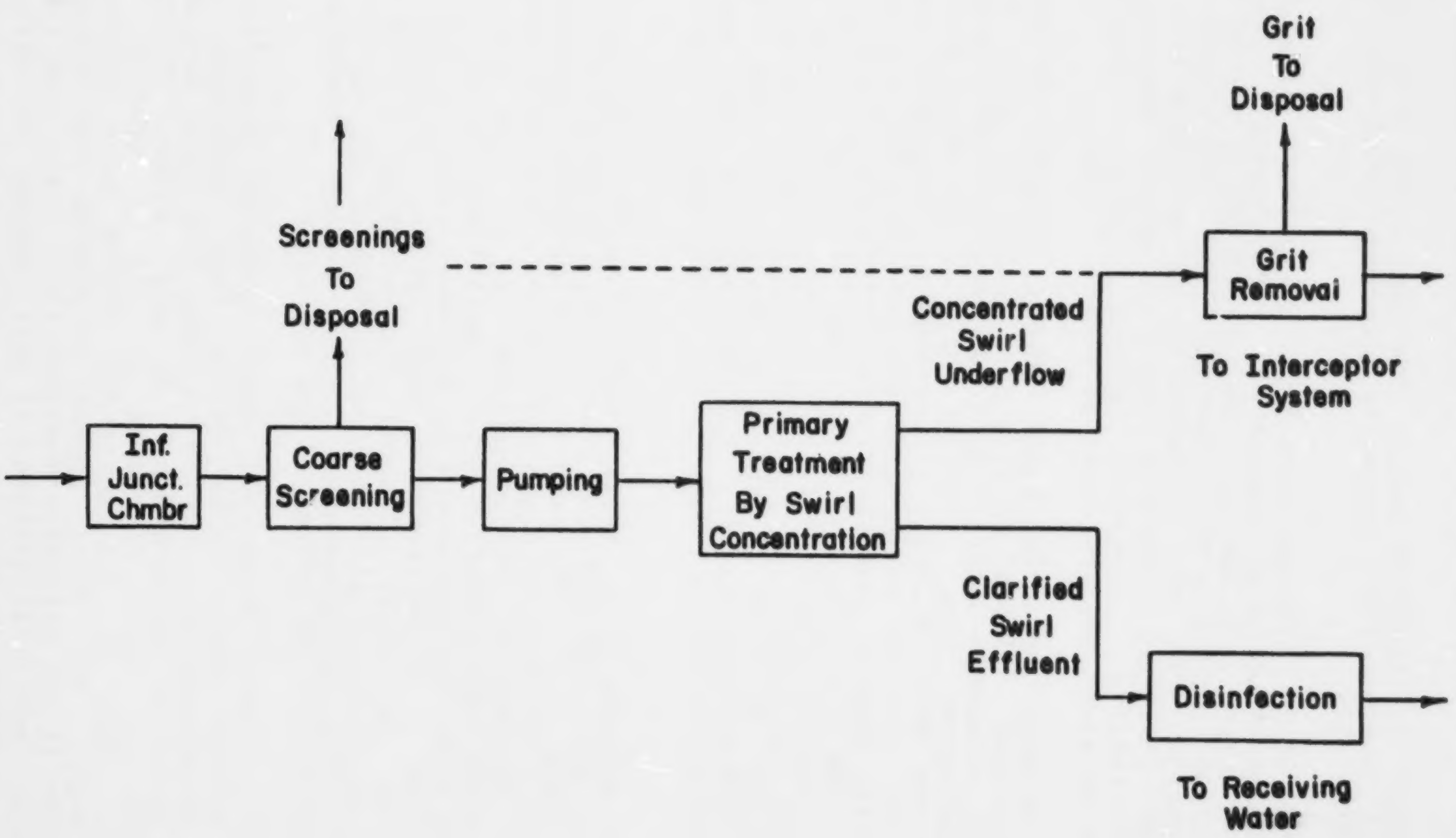

Screenings
To
Disposal
Grit
To
Disposal
Inf.
Junct.
Chmbr
Coarse
Screening
Pumping
Primary
Treatment
By Swirl
Concentration
Concentrated
Swirl
Underflow
Grit
Removal
To Interceptor
System
Clarified
Swirl
Effluent
Disinfection
To Receiving
Water

Figure 5.8 Saginaw study typical CSO primary treatment process scheme.

flow discharged to the foul sewers. The data based on 11 storm events showed average storm hydraulic loading rates as high as 6600 gpd/ft. (8).

The 24-ft diameter dual-functioning swirl regulator/ concentrator constructed in Lancaster, Pennsylvania, under a demonstration grant from the U.S. EPA treats CSO at one of the City`s overflow points to the Conestoga River. Data from the evaluation period (1980-1981) conducted by EDP indicated that the swirl is an efficient treatment device to remove heavier or "first-flush"-related suspended solids (9). Treatment efficiencies exceeded 60% for flows exceeding 20 cfs.

A special cross-sectional sampler capable of taking complete vertical "slices" of flow over an entire pipe cross section in an automatic discrete mode was developed and installed by EDP Technologies, Inc., Hanover, MA. in the 36-in influent conduit to the swirl to obtain representative suspended solids samples. The new cross-sectional sampler and a Manning model 6000 sequential sampler were used during four storm events. An anlysis of the data indicated that suspended solids of samples collected by the new sampler were 6 to 7 times more concentrated than samples collected (at the same instant) by the Manning sampler during the first 10 minutes of the storm's peak "first flush". Concentration factors reduced to 2 to 4 for samples collected mid-event and then down to 0.5 to 2.0 for end-of-event samples.

A summary of the swirl suspended solids removals for five events monitored with the cross-sectional samplers is as shown in Table 5.2. The removal and efficiency percentages associated with the indicated estimated flows are also presented. Removal is defined as the percentage of the influent mass contained in the foul underflow line. Efficiency is defined as the removal minus the percentage of inflow contained in the foul underflow line. These results were derived primarily by inspection of raw data. Overall flow-weighted removal and efficiency calculations are not given because of the approximate nature of flow measurements. Flow meters continuously malfunctioned during the course of the project.

It appears that the swirl provided significant suspended solids removal near design flow conditions. As expected, efficiency deteriorated at lower flow rates because of the flow splitting phenonomenon. In most cases, efficiency exceeded 60% for flows of about 20 cfs or greater. As anticipated, most of the suspended solids were settleable inorganic grit. The swirl appears to provide the degree of treatment that it was orignally designed for, i.e., 90% removal of settleable material (defined as grit having an effective

TABLE 5.2

Lancaster, Pa. 1980 - 1981
Summary of Swirl Regulator/Concentrator Performance

Date of Event	Estimated Flow rate (cfs)	Removal %	Efficiency %
9/10/80	18	70	62
	7	50	39
10/2/80	50	52	47
	7	46	24
	2	80	5
5/10/81	20	12	5
	8	23	5
	6	35	10
7/2/81	55	76	73
	20	86	78
7/20/81 pm	5	41	11
	46	83	80

diameter of 0.35 mm and a specific gravity of 2.61) at design flow of 40 cfs.

Similar in function to the swirl, the helical bend concentrator/regulator appears more practical as in-line devices rather than satellite or off-line devices. Details of the swirl and helical bend regulator/solids concentrators are shown in Figure 5.9.

Screening - Screens have been used to attain various levels of suspended solids and floatable solids removal. Screening methods can be separated into two groups:

1. Course screening used to remove the coarse floatable materials and larger suspended solids. In many cases, coarse screening is used as a pretreatment prior to further treatment to enhance the control process or to protect downstream equipment. Suspended solids removals of 5-10% can be expected.

2. Fine screening used as a primary treatment process. Fine screens include static, drum or rotary screens and also micro-screens. Microstrainers or microscreens have been applied to polishing of secondary effluents at conventional rates of around 110 gpd/sq ft. for 30 micron mesh. Tests of CSO have indicated suspended solids removals of 17-40 percent when using 420 micron screens, 26.6 percent when using 105 micron screens, 10-43 percent when using 70 micron screens and 20-93 percent when using 23 micron screens. Microstrainers are available commercially in a number of configurations. Fine screening devices tend to perform best with low to moderate hydraulic loading rates. Fine screening devices are characterized by relatively high operation and maintenance requirements needed to maintain adequate removal efficiencies and to dispose of collected solids.

Dissolved Air Floatation - Pilot results from treatment of CSO by dissolved air floatation are being used to design a full scale plant in San Francisco. These data inciate suspended solids removal rates of 51 percent at an overflow rate of 4,320 gpd/sq.ft.

High-Rate Filtration - High-rate, multi-media filters have been studied extensively for polishing of sceondary effluents. CSO have been applied to high-rate filters at Washington D. C.; Cleveland, Ohio: Syracuse, Rocheste,r New York, and New York City. Filter media included fiber flass, anthracite and garnet in Washington D. C. studies and anthracite and sand in the Cleveland and Rochester studies. The Syracuse studies also evaluated use of plastic pellets in conjunction with sand and anthracite. The Washington D. C. studies employed flux

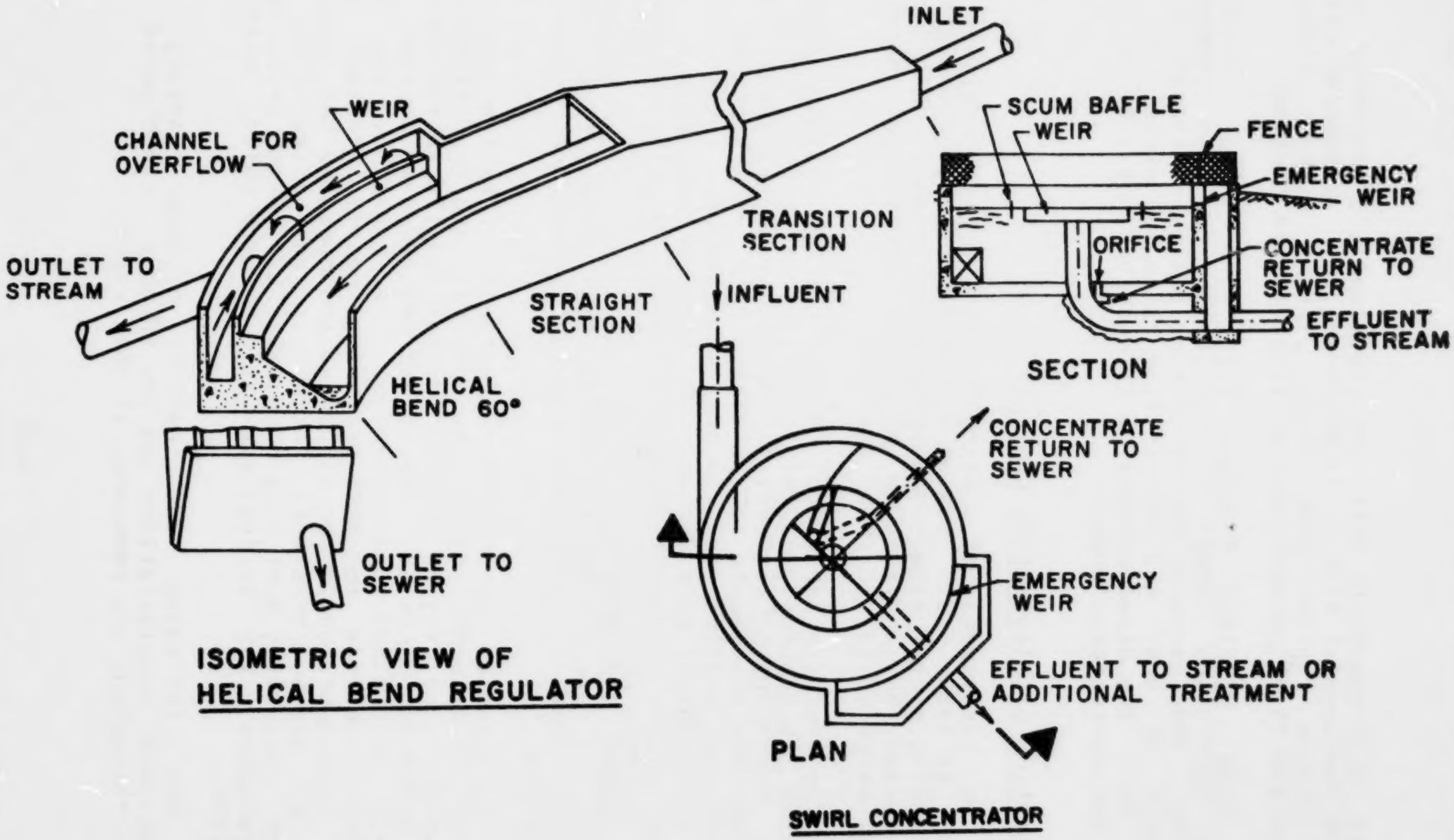

INLET
WEIR
CHANNEL FOR OVERFLOW
OUTLET TO STREAM
TRANSITION SECTION
STRAIGHT SECTION
HELICAL BEND 60°
OUTLET TO SEWER
ISOMETRIC VIEW OF HELICAL BEND REGULATOR
SCUM BAFFLE
WEIR
FENCE
EMERGENCY WEIR
ORIFICE
CONCENTRATE RETURN TO SEWER
EFFLUENT TO STREAM
SECTION
INFLUENT
CONCENTRATE RETURN TO SEWER
EMERGENCY WEIR
EFFLUENT TO STREAM OR ADDITIONAL TREATMENT
PLAN
SWIRL CONCENTRATOR

Figure 5.9 Helical bend and swirl concentrator.

rates of 5-15 gpm/sq/ft while the Cleveland and Rochester work evaluated loadings as high as 25-40 gpm/sq/ft. Suspended solids removal rates ranaged from 30-70 percent without chemical treatment and 85-98 percent with polyelectrolyte treatment.

High Gradient Magnetic Separation - High gradient magnetic separation (HGMS) is applicable for CSO treatment technology. HGMS provides extremely high pollutant removal rates (in excess of 90%) and permits flux rates in excess of 150 gpm/sq. ft. The disadvantages of this technology are the high capital and operational costs.

5.5 Selection of Alternative Controls

An assessment of alternative CSO control options is presented in Table 5.3. System modifications, source controls and collection system management options are broadly encompassed as preventive BMP controls in Table 5.3. Structural controls have been assessed as treatment controls.

The overview management concept for CSO emission control in Saginaw is to provide a fairly moderate to high degree of physical treatment at low overall capital and operational costs. Floatables and settleable solids removal are important from the standpoint of boating and general aesthetics. Settleables removal will control to a degree, CSO heavy metals emission and dissolved oxygen sediment demand. Control of fine suspended solids is more difficult and more expensive but will tend to reduce oxygen demanding substances and turbidity. Turbidity reductions in turn provide for better light penetration, algal productivity (to a limit) and eventually enhanced dissolved oxygen levels.

Acceptable CSO control technologies in Saginaw must be cost effective from both first cost (capital) and operations cost standpoints. The City of Saginaw invests heavily in its various public works departments. The street and sewerage systems are both well maintained and operated. In addition, the City efficiently operates the WWTP providing nearly constant high quality treatment levels and maintains and operates the Hancock Street CSO storage/treatment facility. Any alternative CSO treatment technology must have comparable treatment performance characteristics, but be less expensive to both build, maintain and operate.

The following CSO control options satisfy the aforementioned considerations and are further investigated and detailed throughout the remainder of this report.

	PREVENTATIVE CONTROLS					TREATMENT CONTROLS									
CONTROL TYPE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
SYSTEM MODIFICATIONS						NOT APPLICABLE									
• Regulator Improvements	H	-	H	H	Y										
• Sewer Separation	H	-	L	P	-										
• Improved Utilization of existing facilities	H	-	H	H	Y										
• In-line Storage	H	-	H	H	Y										
• Increase WWTP Utilization	H	-	H	H	Y										
SOURCE CONTROL															
• Street Cleaning	-	M	AM	-	-										
• Solid Waste Management	-	L	AM	-	-										
• Erosion Control	S	M	L	H	-										
• Catchbasin Cleaning	-	L	AM	-	-										
• Roof Storage	S	-	P	-	-										
• Parking Lot Storage	S	-	M	L	-										
• Natural Flow Attenuation	S	-	L	-	-										
• Porous Pavement	S	-	P	-	-										
COLLECTION SYSTEM MANAGEMENT															
• Polymer Injection	S	S	-	-	-										
• Sewer Flushing	-	S	P	-	-										
• Sewer Cleaning	-	S	AM	-	-										
STRUCTURAL CONTROLS	NOT APPLICABLE														
• Storage/treatment facilities						M-H	M	H	M	L	L	H	M	C	Y
• Swirl Regulator/Concentrators						H	L-M	H	L-M	N	S	L	L	C	Y
• Screening (Coarse)						M	L	L	M	N	N	L	L	-	-
• Screening (Fine)						H	M	H	M	N	S	M	H	C	-
• Dissolved Air Floatation						H	M	H	M	L	S	M	H	C	-
• High-rate Filtration						H	M	H	H	M	L	H	H	C	-
• High gradient Magnetic Separation						H	VH	VH	VH	H	H	H	H	C	-

KEY: Preventative

VH = Very high
H = High
M = Moderate
L = Low
S = Slight
N = None
AM = Already maximum
P = Poor
Y = Desireable control option (continue investigation)
C = Meets treatment requirements

Treatment Code (% Removal)

95 - 99 (VH)
70 - 95 (H)
40 - 70 (M)
10 - 40 (L)
less than 10

COLUMN HEADING KEY:

1: CSO Volume/Load Reduction Potential
2: CSO Load Reduction Potential
3: Physical Feasibility
4: Cost Feasibility
5: Further Investigation
6: Floatables Removal
7: Suspended Solids Removal
8: Settleables Solids Removal
9: Organics Particulate Removal
10: Colloidal Removal
11: Nutrients Removal
12: Capital Costs
13: Labor Intensive - Maintenance
14: Meets Treatment Requirements
15: Feasible for Saginaw Further Investigation

TABLE 5.3 OVERVIEW OF DESIREABLE CSO CONTROLS FOR SAGINAW

A. System Modification

- regulator improvements
- enhanced hydraulic conveyance
- increased in-system storage

B. Structural Controls

- storage/treatment facilities
- swirl regulators/concentrators

Fine screening (rotary/micro screens), dissolved air floatation, high-rate filtration and high gradient magnetic separation are technologies that meet CSO emission reduction expectations and would attain perceived water quality goals, but they would not satisfy Saginaw's need for cost-effective capital and more importantly, operationally inexpensive solutions.

The combination of "system modification" options is hereinafter referred to as the BMP plan.

CHAPTER 6
COMBINED SEWER MODELING

6.1 Foreword

The Saginaw CSO Management Model, developed by EDP, is an inexpensive and flexible tool for preliminary planning and sizing of CSO facilities. The model provided a means by which the CSO problem could be analyzed and introduced time and probability to the quantity, frequency and duration of overflow events by using actual rainfall data collected over a long period of time. It was used to screen alternatives based on varying storm conditions.

The purpose of this chapter is to outline the modeling tools, and their utilization in the development of the various proposed and implemented control programs for treating combined sewer overflows. A comprehensive mathematical computer simulation program was developed to help determine overflow discharges and pollutant concentrations as a result of hypothetical occurrence of various size design storms. Such a model not only provided an accurate representation of the physical sewerage system, but also provided an opportunity to determine the benefit of proposed pollution abatement alternatives.

The value of modeling lies in the ability to quickly predict the outcome of dependent variables as a result of hypothetical modification of the system. These quick projections have allowed for more time-efficient evaluation of abatement alternatives formulated to meet established goals, concepts, and criteria. Through this process the best alternatives were isolated for further cost-benefit analysis.

Model development and calibration was performed over three time periods. Initially, during 1979-1980, EDP developed the basic model which was used to prepare the CSO Facility Plan, (Phase 1 of the project). During 1983 (Phase 2) further modeling was performed to detail the Reduced BMP program. During that time minor model modifications were performed to optimize calibration based on available data. Phase 3 of the modeling work was performed during 1984 before and after implementation of the Reduced BMP plan. An extensive (Fall, 1984) field investigation provided the necessary data base to further calibrate the model using improved knowledge of rainfall/runoff characteristics, regulator performance and pump station operations.

The culmination of the preceding efforts provides a documented assessment of appropriate technologies for abating combined sewer overflows. This is a recommended plan of action resulting from a complex analysis of a complex problem with a wide range of variables and associated costs and benefits.

Runoff volume and quality are predicted on the basis of hourly rainfall. dry weather sewerage flow (and infiltration), street washoff. dry weather sewerage and deposits. Regulator operations are simulated for each of 31 catchment areas previously defined. Resulting overflows to the river are retained for statistical analysis and flows to the interceptor passed into the interceptor model. The interceptor model simulates the operation of the intercepting system including overflow and quality into the pump stations on the interceptor, the river crossing flows, and flows and quality to the waste water treatment plant. Treatment operation(s) are simulated both at the existing WWTP and at any overflow point upstream. The costs and effectiveness of the various options available to Saginaw for stormwater management are calculated and summarized for any given combination of alternatives. The entire model operates on an hourly time increment.

Details of the runoff prediction computations are given in Section 6.2. Regulator and in-line storage operations are discussed in Section 6.3 and 6.4, respectively. Interceptor model details are given in Section 6.5. West Side Interceptor river crossing details are given in Section 6.6. Water quality modeling details of the sewerage system are discussed in Section 6.7. WWTP simulation details are given in Section 6.8. Discharge calibration details are given in Section 6.9. Phosphorous modeling details are presented in Section 6.10.

6.2 Runoff Prediction

An overview of the simulation model runoff volume and water quality estimation procedure for each of the 31 catchment areas is shown in Figure 6.1. Runoff volume for each catchment area is calculated for each hour of rain using the runoff coefficients, the catchment area and average hourly rainfall intensity.

For most catchment areas the runoff is considered delivered to the regulator chamber during the same hour of recorded rain occurance. For the Weiss and Fourteenth Street catchment areas, a one hour time delay is used for the section of the catchment area estimated to be over an hour away during full pipe flow conditions. Hourly dry weather sewerage flow per catchment area is added to the runoff flow. For the spring months of April - June. infiltration estimates per catchment area are

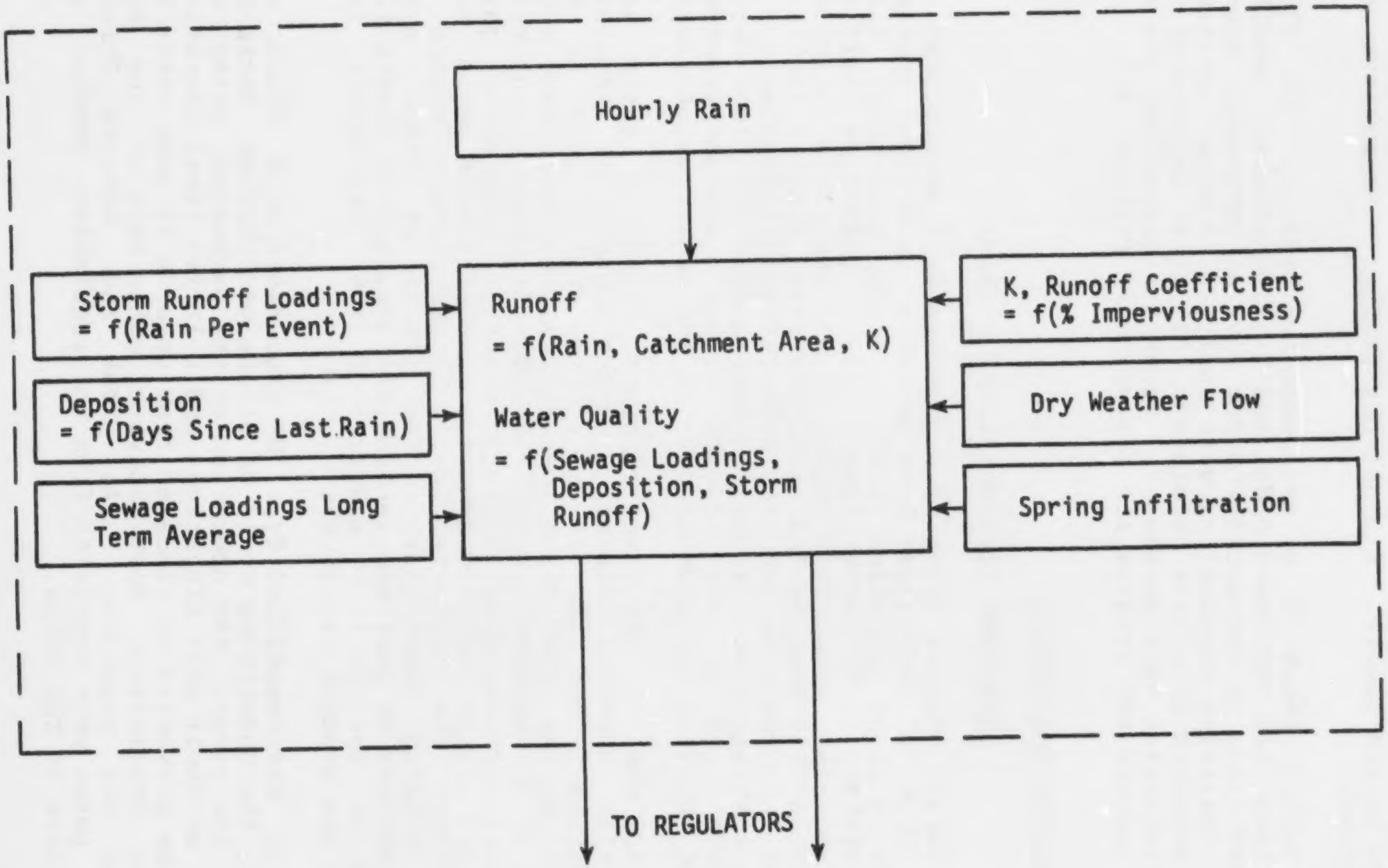

Hourly Rain
Storm Runoff Loadings
= f(Rain Per Event)
Deposition
= f(Days Since Last Rain)
Sewage Loadings Long
Term Average
Runoff
= f(Rain, Catchment Area, K)
Water Quality
= f(Sewage Loadings,
Deposition, Storm
Runoff)
K, Runoff Coefficient
= f(% Imperviousness)
Dry Weather Flow
Spring Infiltration
TO REGULATORS

Figure 6.1 Catchment area modeling.

also added to the runoff. Diurnal dry weather variations are not considered.

During Phase 3 of the modeling effort, EDP field investigations and continued model calibration efforts resulted in increased times of concentration for several catchment areas. Regulator operation simulations were modified to more suitably match the expanded data base developed. All other aspects of the Phase 1 evaluation were unchanged. Regulator operations pre and post the Reduced BMP Program are discussed in Section 6.3.

6.3 Regulator Operation:

1979-1981 Considerations - Phase 1

Three general types of regulators were encountered in Saginaw. Type "B" regulators have variable underflow openings altered by a float operation (Brown & Brown mechanical float operated systems). As water levels rise, underflow orifices close. Type "A" regulators have fixed underflow orifices, generally about 1/2 foot by 1/2 foot. The third type of regulator encountered are the five modified Type "B" regulators located in the Hancock Street area. These regulators were modified during the implementation of the Hancock Street storage/treatment facility to increase the closure rate of the underflow orifices.

At two of the five locations backwater gates were replaced with automatic sluice gates set to open at regulator storage depths above the level of the spill weir to the interceptor. At the other three locations sluice gates were added downstream of the backwater gates. Increased closure of underflow orifices will decrease flow to the interceptor. The sluice gates captured and stored combined sewerage that otherwise would have passed through the backwater gates to the river (except in extreme flooding conditions). The intent of these prior modifications was to increase in-line storage, and to divert flow in excess of the in-line storage to the interceptor (and eventually the Hancock facility).

In the unmodified Type "B" regulators high flows not handled by the underflows would be discharged through backwater gates to the river. The operation of the backwater gates was dependent on their seat elevations and relative river elevation, and could be prevented or hindered from opening in some cases for high river conditions. Most backwater gates were in the same chamber as the regulator. At Cronk and Genesee Streets backwater gates were separated from the regulation chamber and were proximate to the river.

Regulator performance data was limited to the operation of the modified Type "B" regulator chambers in the Hancock Street area on the West Side. Strip charts recording stage of backed-up combined sewage as a function of time during storm events were available for five regulators located at Mackinaw Ave., Adams Street, Throop Street, Hayes Street and Remington Street. The recording devices at the Mackinaw regulator was defective and was eliminated from the analysis. These data were analyzed to determine dynamic operation of the regulator float devices during rain events. Zero reference points on the strip charts were impossible to determine and the rate of water level increase during rising portion of the hydrographs were not considered to be reliable since discharge over the weir levels to the interceptor may have occurred which would not have been noted on the strip charts. The fall of water level over time during the recession of the storm event (drainage/ in-line storage) was considered accurately recorded. Underflow discharges, as a function of time were estimated using the strip charts rate of depth change and estimated storage volume changes associated with the depth change. Underflow rates as a function of stage for the four regulators are summarized in Figure 6.2.

System operation data were not available for the other Type "B" regulators. Underflow dynamics were estimated on the basis of dimension measurements and visual inspections by EDP field engineers during storm events. It appeared during the EDP inspection program that the float mechanisms were slow in responding to rising water levels, and it was speculated that many underflow orifices did not appreciably close during high water conditions. Underflow drain pipes leading to the interceptor were determined to be large enough to handle maximum underflow discharge.

The dynamics of existing regulator operation were modeled as depicted in Figure 6.3 for the hourly averaged runoff flows. All underflows were calculated as a function of depth. The available data for the Hancock area regulators were used to generate functions for simulation . Underflow in these cases decreased as depth of flow increased. Other regulator underflows in the system were calculated on the basis of a constant area orifice discharge to free space given by equation:

underflow = area x square root (2gh)

where h is the depth of flow above the orifice. It was determined that connecting lines to the interceptor were large enough to handle high flows, so the limiting aspect of flows to the interceptor was the operation of the orifice openings.

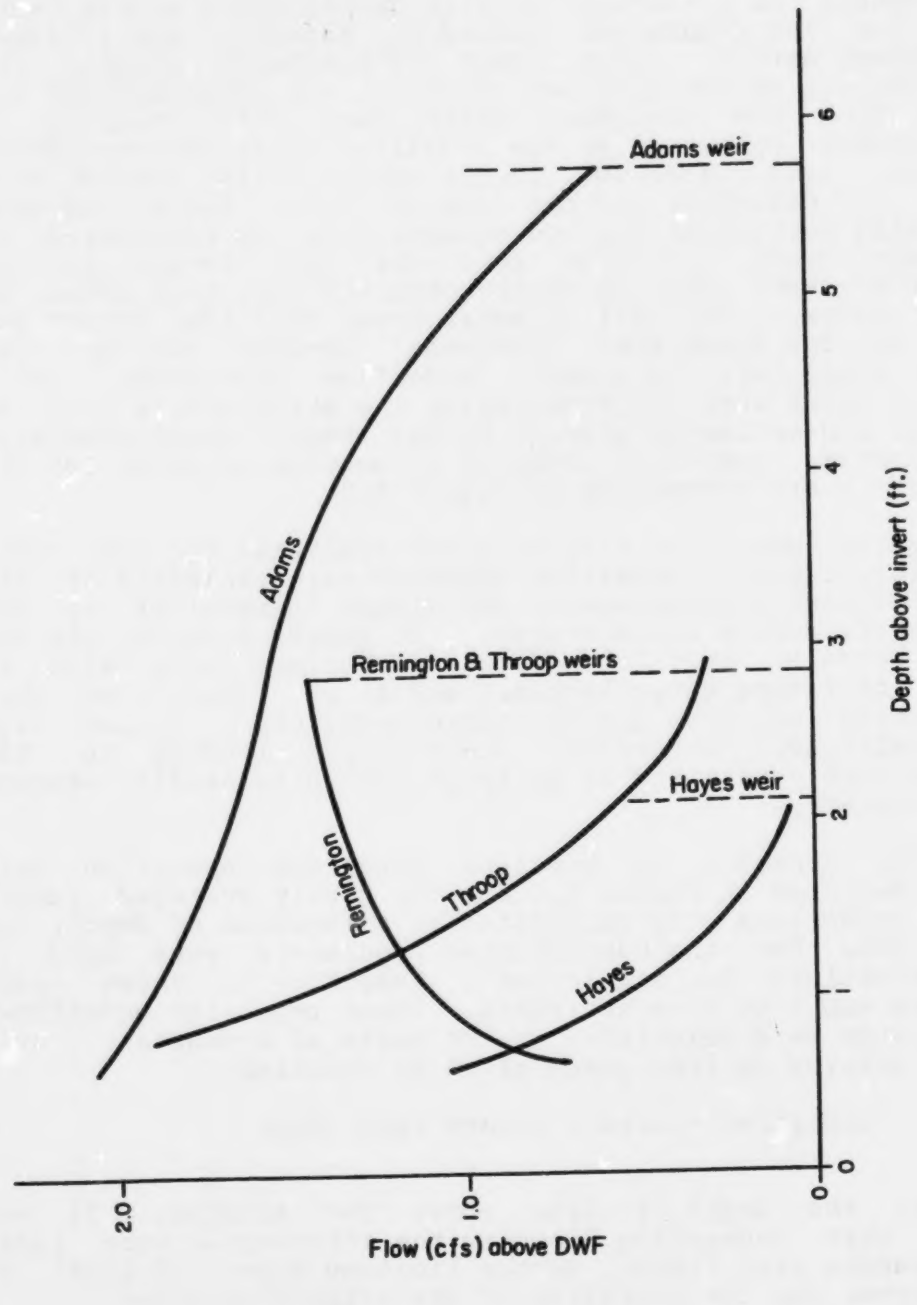


Figure 6.2 Discharge / water depth curves, Hancock area regulators.

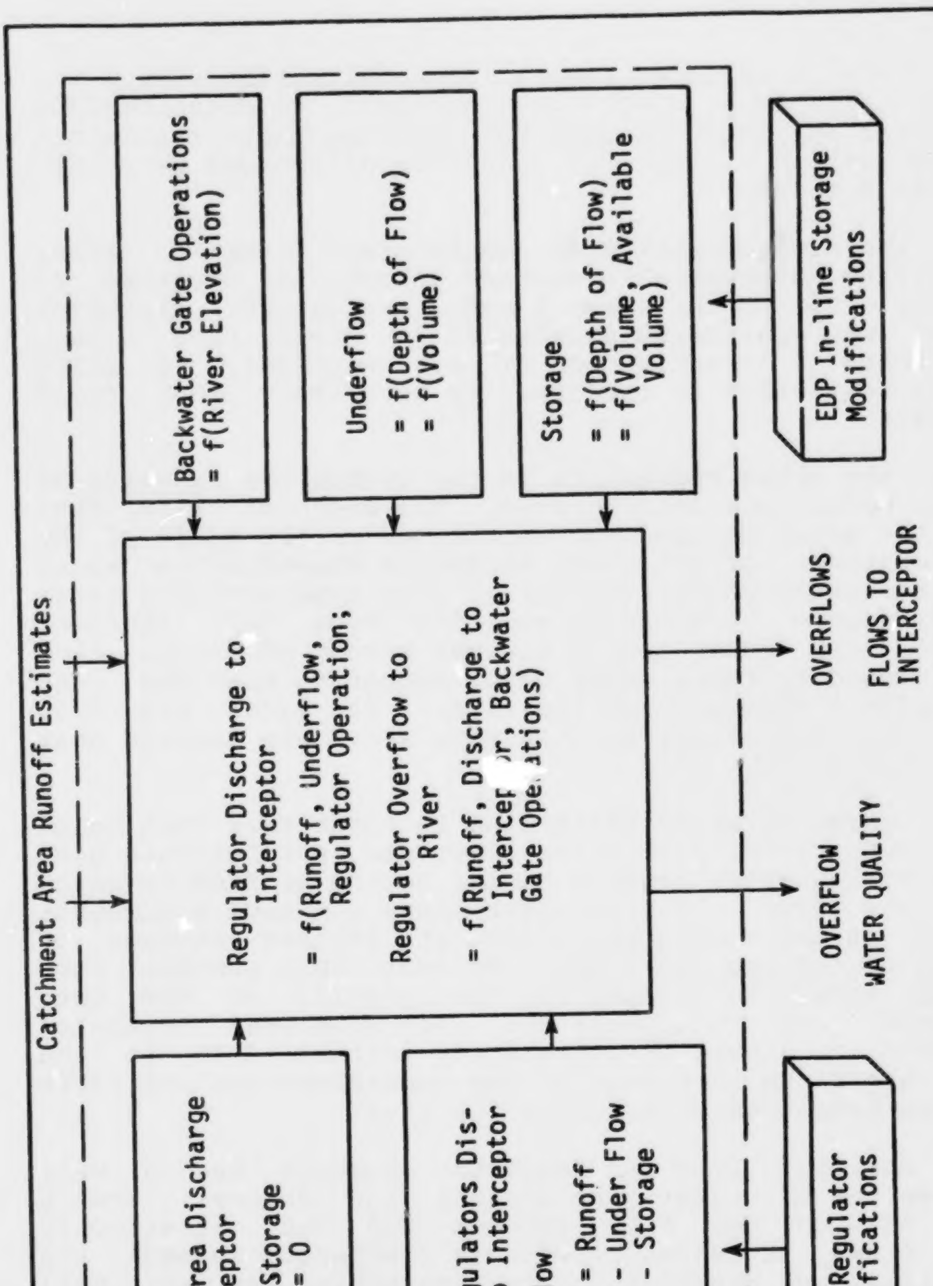

Catchment Area Runoff Estimates
Hancock Area Discharge To Interceptor
= Runoff-Storage
Overflow = 0
Other Regulators Discharge to Interceptor
= Underflow
Overflow = Runoff - Under Flow - Storage
Regulator Discharge to Interceptor
= f(Runoff, Underflow, Regulator Operation;
Regulator Overflow to River
= f(Runoff, Discharge to Interceptor, Backwater Gate Operations)
Backwater Gate Operations = f(River Elevation)
Underflow
= f(Depth of Flow)
= f(Volume)
Storage
= f(Depth of Flow)
= f(Volume, Available Volume)
EDP Regulator Modifications
EDP In-line Storage Modifications
OVERFLOW
WATER QUALITY
OVERFLOWS
FLOWS TO INTERCEPTOR

Figure 6.3 Schematic of regulator operation.

As a result of later simulation model calibations, the assumption of free-fall discharge conditions through maximum regulator orifice openings was used for modeling those regulators where data was lacking. Type "A" regulator discharges were also estimated in this manner.

For the five Hancock area regulators, flows in excess of the underflow and available storage volume were diverted to the interceptor. During the Phase 1 and 2 evaluations discharge to the river was considered eliminated in these five cases. During the Phase 3 investigation it was necessary to allow extreme flows to outflow to the river to accurately match field and modeled data.

For the other regulators in the system the dynamics of the backwater gates were incorporated. The depth of flow that would cause the gates to open was calculated on the basis of the gate seat elevation, or the river elevation depending on which was higher. Backwater gates are typical flap-type swinging gates (with the exception of the five modified Type "B" regulator chambers). In most cases only a minimal amount of storm flow increase is needed to raise water depth enough to open the flap gates and cause a discharge to the river. The gates are also sensitive to river elevations as most gate seats are located near average river elevation.

The Weiss Street gravity line is about five feet below average river elevations. High river flows can restrict this gate operation and subsequently require higher depths of flow to cause discharges to the river. Varying river stage can have a somewhat random effect in increasing the amount of in-line storage of storm flow and decreasing overflows. An additional one-half foot elevation was added to account for the dynamics of the gate movement itself. and to insure that the simulated regulator operation would at least divert all dry weather flow to the interceptor. Any flows in excess of the underflows and available storage volumes were then diverted to the river.

The modified Type "A" regulator chambers had a weir placed between the backwater gate and the flow channel, and a lower weir between the flow channel and the interceptor connection. Flows in excess of what is discharged through the underflow orifices or stored in the system behind the weir will first overflow the lower weir and be directed to the interceptor. Only extremely high flows will overflow the weir separating the backwater gate portion of the chamber and be directed to the river. This weir arrangement was considered by EDP to be both the least expensive and most available method of limiting overflows at any of the unmodified regulator chambers in the system.

Regulator Operation: 1984 Considerations - Phase 3

Under the complete BMP Plan it is envisioned that all regulator chambers be converted to act in a similar storage mode as the Hancock area modified Type "B" regulators, but with the addition of constant underflow discharge devices. As stated the Type "B" regulators earlier modifed as part of the Hancock Street detention improvements had been altered to cause the underflow orifices to close rapidly, limiting flows to the interceptor and thus committing the first flush of a rain event to in-line storage created at the chamber. EDP considered it more advisable to maintain higher underflow discharges to the interceptor, limited only to what the interceptor can maximally handle. Operation in this manner will commit the first flush of a rain event, generally carrying the highest concentration of pollutants, to the interceptor, and eventually to the WWTP. This mode of operation is preferable to capturing the first flush in system storage which may become overflow at a later time if the rain continues. Furthermore, this mode can be accomplished with constant discharge devices that are more reliable and easier to maintain than the present float mechanisms. These devices will be discussed further in Chapter 7.

For the five Hancock area regulators, CSO in excess of the underflow and available storage volume were diverted to the interceptor. River discharge was considered eliminated in these five cases except during extreme events. Underflows were set equal to approximately six times dry weather flow unless higher underflows were needed to handle the dry hour discharges of in-line storage pools created further upstream. Weirs were adjusted to create in-line storage at the regulator chamber, as appropriate. With the exception of Fraser, Webber, Fourteenth and Weiss Street regulators, all but extreme excess flows were diverted to the interceptor. Fraser, Webber, Fourteenth and Weiss Street regulators were allowed to overflow when runoff flows were in excess of underflows and available in-line storage volumes.

Under the Reduced BMP plan all West Side regulators were modified to act in the preferred mode and modeled as such in the Reduced BMP plan simulations.

6.4 In-line Storage

One concept of this study is to maximize the use of existing facilities such as in-line storage already available in the system conduits. The in-line storage concept for the complete BMP plan consisted of alteration of 31 regulator/backwater chambers for in-line storage purposes and construction of 14 flow control points upstream of the appropriate regulator chambers.

The purpose of the upstream control points was, in some cases, to avoid the steeply sloping inverts on the West Side near the river and, in other cases, to simply add a second level of flow control in addition to control at the regulator. Regulators on the West Side are typified by conditions where the influent trunk sewers are on a steep incline to the regulator chambers which then level-off not far upstream. Far greater storage volumes could be generated for these trunk sewers if the flow control structures were located upstream where the invert slopes are less steeply inclined. Control structures in the Fourteenth and Webber areas were moved upstream to avoid possible flooding due to low ground surfaces. The structure for the Fraser area was moved upstream to avoid possible negative interaction with privately owned sewers in that area.

During the Phase 1 modeling, in-line storage potential was assessed. Pertinent details of the sewer system were transferred from the sewer plans to a city-wide base map. All pipes equal to and greater than 36 inches were outlined on the base map. Pipe sizes, invert elevations and manholes were noted. Contours of ground surface elevation (2-foot interval) were added as an overlay to the resulting base map.

An interactive computer program was developed during Phase 1 to calculate in-line storage that would result by flow restrictions at any point in the system. Area of flow as a function of depth was calculated for all typical pipe cross-sections in the Saginaw system, including: circular, oval, egg and horseshoe-shaped sections. The in-line storage program used these area functions to calculate volumes stored behind any given weir height associated with the level pool in static conditions. The resulting storage volumes were conservative in that the dynamics of flow had been ignored in the level pool approximation. Invert slopes were also included in the calculations allowing volumes over changing slopes to be accurately calculated.

One new aspect of the Phase 2 modeling was a detailed review of in-line storage potential of all piping within the system. Where potential backup could occur it was included regardless of pipe size. A more accurate estimate of the actual potential storage volume resulted by inclusion of pipes under the previous 36 inch limitation. Storage volume increases due to this procedure varied throughout the city. In some areas the trunk lines are shallow and most lateral lines are subject to backwatering, whereas in others, lateral lines feed deep trunk lines at drop manholes and backwatering would be nonexistent.

Flow dynamics' implications to storage were also considered in lieu of the previous static pool level volume estimation. Rainfall/runoff data generated in the Phase 1 evaluation were utilized in Phase 2 to predict nominal flow rates within the piping system. Flow was determined as a function of overall area drainage and the fractional portion of area upstream of each pipe segment. Friction-induced head loss calculations were performed and accumulated with distance upstream to formulate a hydraulic profile within the system. Use of these flow dynamics also resulted in a more accurate estimate of the actual potential storage as the volume of storage increases beyond the static, level pool model.

Hydraulic profiles and resultant storage volumes at any time were shown to be a function of flow rate (a direct result of rainfall intensity). A nominal intensity of 0.1 inches per hour was selected for final analysis. Estimates of storage volume also varied throughout the city dependent upon the relative pipe sizes to areas served. Large areas generate greater runoff volumes than small areas such that for any given pipe size frictional head loss and resultant hydraulic profile levels were greater implying fuller pipe conditions and greater storage potential.

Table 6.1 shows the relative increases in estimated storage volume due to the various aforementioned considerations for the Throop Street catchment area. A substantial volume increase of 14.8% is attained by considering all pipe sizes, with an additional 21.7% volume increase based on manhole volumes and flow dynamics as compared with earlier calculations. These values are based on a backwatering weir levels at the pipe crown elevations. Actual volume increase percentages will vary with weir height at this location and will vary widely at all other locations.

Although the revised method of storage calculation tends to increase the predictable storage volume at each location, the final storage volume may not increase above the 1981 predicted amount. Exact design hydraulic evaluation of each chamber reduced the final weir elevations in several chambers for the partial BMP plan implementation thereby reducing anticipated storage volumes. Similar reductions will probably occur at other regulators during final complete BMP implementation of East Side regulators such that overall storage volumes provided may be similar to those estimated during 1981.

The potential for in-line storage was estimated using the in-line storage computer program for selected sites (regulators and further upstream control points). In the selection of upstream sites no point was considered for which ground elevation was less than ten feet above the resulting water elevations after considering weir heights and flow hydraulics at

TABLE 6.1

TYPICAL IN-LINE STORAGE VOLUME COMPARATIVE ANALYSIS

LOCATION: THROOP STREET - WEIR 5.5 FEET ABOVE INVERT

TOTAL STORAGE PIPES 36" AND ABOVE NO FLOW DYNAMICS NO MANHOLE VOLUME	18000 FT^3
TOTAL STORAGE ALL PIPING NO FLOW DYNAMICS NO MANHOLE VOLUME	20662 FT3
TOTAL STORAGE ALL PIPING WITH FLOW DYNAMICS NO MANHOLE VOLUME	24581 FT^3
TOTAL STORAGE ALL PIPING NO FLOW DYNAMICS WITH MANHOLE VOLUME	21233 FT^3
TOTAL STORAGE ALL PIPING WITH FLOW DYNAMICS WITH MANHOLE VOLUME	25155 FT^3

SUMMARY

ANALYSIS MODE	VOLUME INCREASE	PER-CENT INCREASE
ALL PIPING*	5662	14.8
FLOW DYNAMICS**	3919	19.0
MANHOLE VOLUME**	571	2.7
FLOW DYNAMICS PLUS MANHOLE VOLUME	4493	21.7

* COMPARISON OF ALL SYSTEM PIPING TO PIPES OVER 36 INCHES

** BASED ON ALL SYSTEM PIPING

peak rates to insure later plan implementation would not flood basements. Also, no point was considered that would cause backup beyond a point where trunk sewer lines intersected. The intention of this limitation was to avoid altering the flood relief patterns for which the relief system had been originally designed and built, and to avoid diverting flows to different regulators through the bifurcation points.

Empirical functions were developed at each candidate in-line storage control point relating stage (of back-up) to storage volume. These functions were prepared by using the interactive computer program to determine in-line storage estimates for several plausible elevations and then curve fitting the results. These empirical functions were then utilized in the simulation model.

Pre BMP in-line storage conditions were analyzed based on average river conditions with the gate-river interactions determining the extent of storage in many cases. The total pre BMP storage volume was calculated to be 327,000 cubic feet. The total potential capacity for system storage including the two aeration tanks at the WWTP is estimated to be 3.93 million cubic feet. or the equivalent of approximately seven of the Hancock Street sized storage basins. In-line sewer system storage including the interceptors account for 58 percent of the total. Storage estimates for individual catchment areas are presented in Table 6.2 Storage estimates are provided for pre BMP conditions and for the final BMP plan as determined by EDP.

Exact storage volumes for locations to be modified under the complete BMP program will be dependent upon final selection of weir elevations. This selection will be based on detailed hydraulic evaluation similar to those performed for the regulators modifed under the reduced BMP plan and detailed in Chapter 7.

6.5 Interceptor Model

The interceptor model is summarized schematically in Figures 6.4 and 6.5. Figure 6.4 is an overview of the model while Figure 6.5 illustrates the computational procedure.

The interceptor model divides the interceptor system into four sections. Each section is treated as a quasi independent "tub." A mass balance is performed to account for influents, effluents and pump station overflows. Dynamics are incorporated to model the river crossing and to calculate discharge from one section to the next.

TABLE 6.2

IN-LINE STORAGE POTENTIAL

Location of Flow Control Device	Estimated "Pre BMP" Storage (Low River) (1000 Cubic Feet)	ESTIMATED "BMP" Storage Capacity (1000 Cubic Feet)
1. Downstream		
East Side		
14th	25	25*
First	2.4	4.7
Carlisle	5	15
Fitzhugh	36	70
Johnson	1.3	27
E.Genesee	0.7	6.9
Federal	4.5	25
Janes	4.5	125
Millard	1.2	6.7
Thompson	35.2	63
Hoyt	1.4	5.3
Emerson	2.4	5.8
Holden	1.4	14
McCoskry	66	110
Holland	2.7	22
Webber	38	38

TABLE 6.2 INLINE STORAGE POTENTIAL (CONTINUED)

Location of Flow Control Device	Estimated "Pre BMP" Storage (Low River) (1000 Cubic Feet)	ESTIMATED "BMP" Storage Capacity (1000 Cubic Feet)
West Side		
Weiss (ave. river)	23	66
Cronk	36	54
W. Genesee	1.0	19.0
Remington	3.8	2.5
Throop	3.8	4.1
Hayes	3.3	2.6
Adams	51	21
Mackinaw	1.9	1.6
Fraser	24	24
2. Upstream		
14th area:		
at 17th & Perkins	-	253
at 14th & Normal	-	106
Webber area:		
at River St. & Cambrey	-	191
at Birch & Harris	-	153
at Troy & Boxwood	-	42
Fraser area:		
at Vermont & Salt	-	191**

TABLE 6.2 IN-LINE STORAGE POTENTIAL

Location of Flow Control Device	Estimated "Pre BMP" Storage (Low River) (1000 Cubic Feet)	ESTIMATED "BMP" Storage Capacity (1000 Cubic Feet)
Mackinaw area:		
at Hamilton & Williams	-	92
at Gratoit & Michigan	-	125
at Mackinaw & Michigan	-	54
Adams area:		
at Jackson & Mason	-	60
at Cass & Woodbridge	-	67
Hayes area:		
at Hayes & Michigan	-	8
Cronk area:		
at Union & Delaware	-	133
Totals	327	2655
Interceptor Storage	770	770
WWTP	500	500
Total Storage***	1597	3925

* Storage control was not located at regulator chamber plan in lieu of upstream control

** Included as part of reduced BMP Plan underflow control modification

*** Does not include Hancock street storage basin

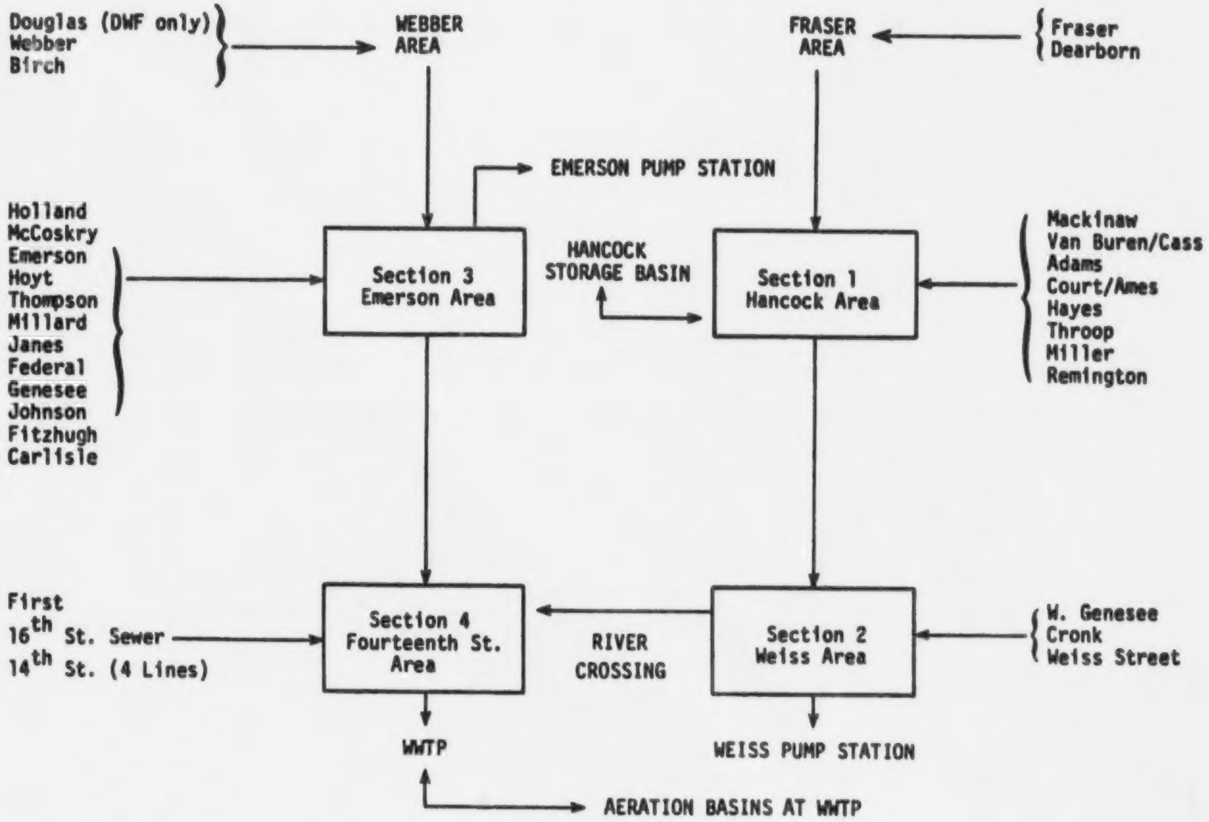

Douglas (DWF only)
Webber
Birch
WEBBER AREA
FRASER AREA
Fraser
Dearborn
EMERSON PUMP STATION
Holland
McCoskry
Emerson
Hoyt
Thompson
Millard
Janes
Federal
Genesee
Johnson
Fitzhugh
Carlisle
Section 3 Emerson Area
HANCOCK STORAGE BASIN
Section 1 Hancock Area
Mackinaw
Van Buren/Cass
Adams
Court/Ames
Hayes
Throop
Miller
Remington
First
16th St. Sewer
14th St. (4 Lines)
Section 4 Fourteenth St. Area
RIVER CROSSING
Section 2 Weiss Area
W. Genesee
Cronk
Weiss Street
WWTP
WEISS PUMP STATION
AERATION BASINS AT WWTP

Figure 6.4 Interceptor model overview.

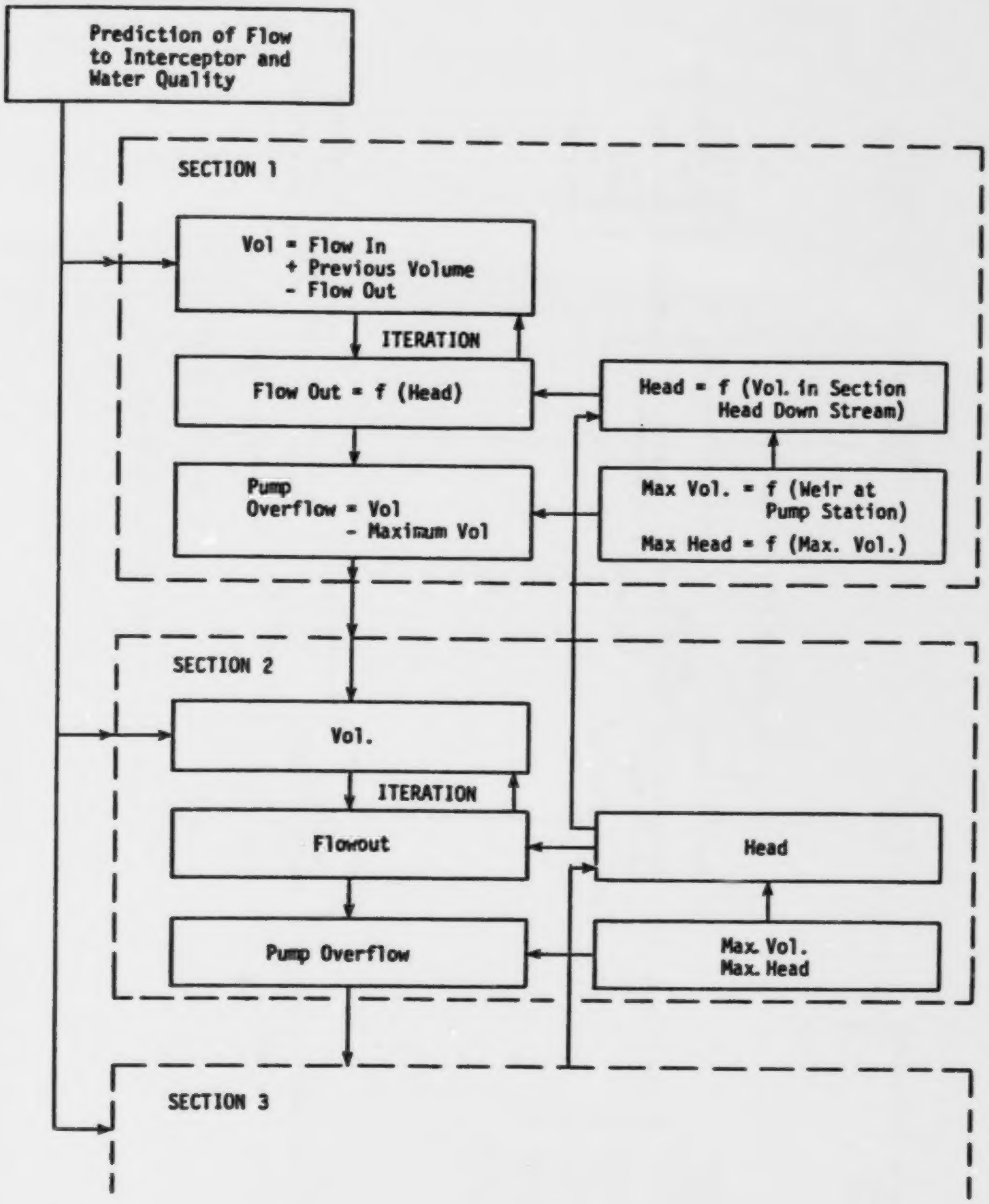

Prediction of Flow to Interceptor and Water Quality
SECTION 1
Vol = Flow In + Previous Volume - Flow Out
ITERATION
Flow Out = f (Head)
Head = f (Vol. in Section Head Down Stream)
Pump Overflow = Vol - Maximum Vol
Max Vol. = f (Weir at Pump Station)
Max Head = f (Max. Vol.)
SECTION 2
Vol.
ITERATION
Flowout
Head
Pump Overflow
Max. Vol.
Max. Head
SECTION 3

Figure 6.5 Interceptor model computations.

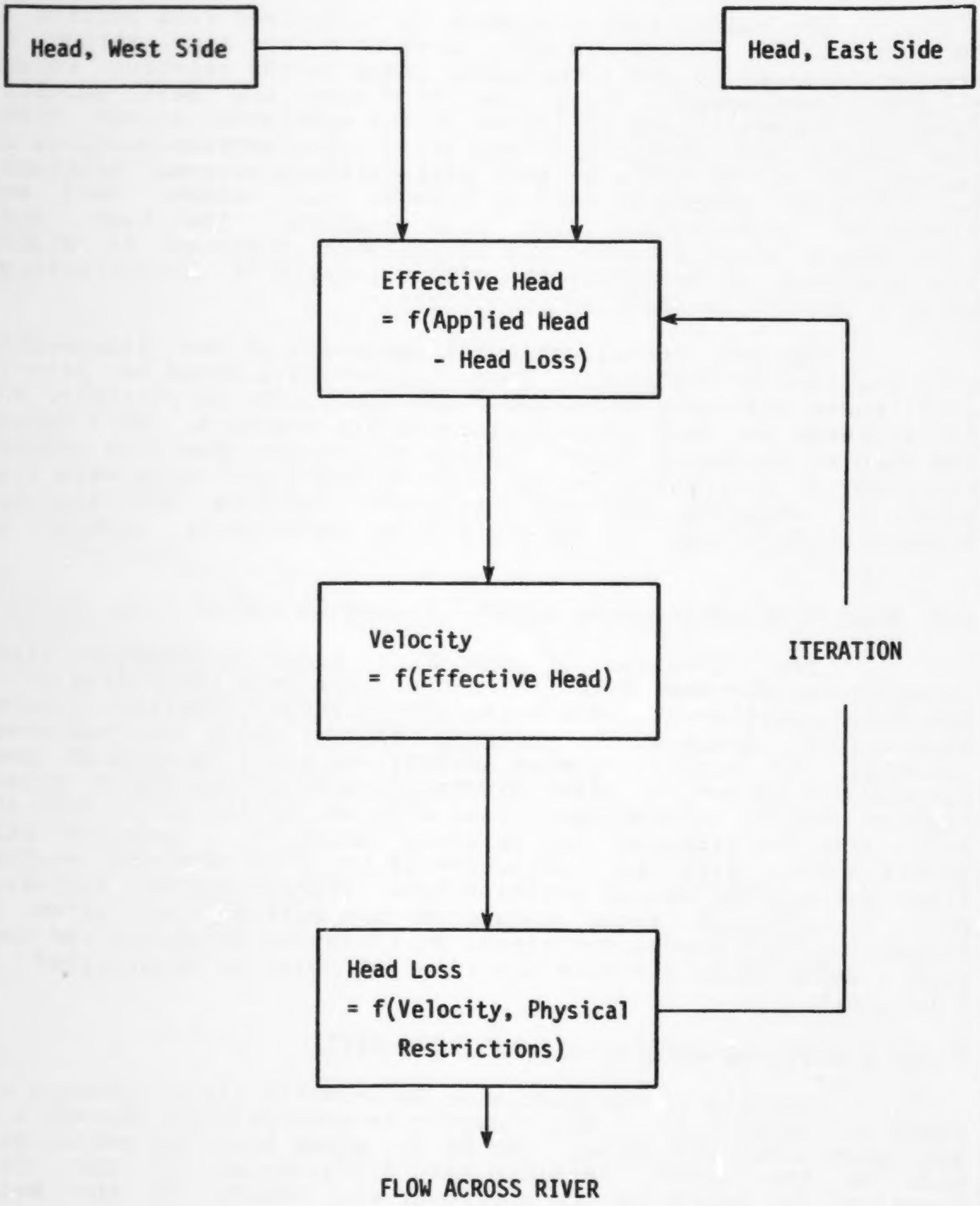

Head, West Side
Head, East Side
Effective Head
= f(Applied Head
- Head Loss)
Velocity
= f(Effective Head)
ITERATION
Head Loss
= f(Velocity, Physical
Restrictions)
FLOW ACROSS RIVER

Figure 6.6 West Side river crossing model.

The calculation procedure is to proceed from section 1 to section 4. Volume is calculated from the time stream of regulator flows to the interceptor added to the existing volume in the interceptor, minus the flow into the next section. Overflow is any volume in excess of the available volume after the volume calculations are completed. Flow between sections is calculated on the basis of head differentials between sections. An iteration scheme is used to balance the volume, head and discharge calculations for each section. The West Side Interceptor river crossing simulation model depicted in Figure 6.6 operates on basically the same principle of head/discharge balance through an iteration procedure.

Options for altering the operation of the interceptor were included in the model. Pump station weirs could be altered to increase available interceptor storage volume if possible, and to increase the head differential between sections. This option was used at the Weiss Street Station to increase the flow across the river. Alterations at the WWTP as described below were also built to interact with the interceptor model by altering the allowable discharge to the plant from interceptor section 4.

6.6 West Side Interceptor River Crossing Considerations

The junction of the West Side Interceptor river crossing and the East Side Interceptor has been identified as a hydraulic bottleneck. The Weiss Street Pumping Station relieves backwater/surcharge at the crossing and is a major combined sewer overflow. It is clear from an analysis of the frequency of pump operation and pumping plant discharge rates at the Weiss Street Station during rain events, that flow across the river into the East Side Interceptor is severely limited. Coupling this understanding with the objective of handling more wet weather flow through the use of aeration tank storage and/or increased WWTP flow-through rates suggested consideration be given to increasing flow across the river. An iterative procedure was used during modeling to simulate the river crossing as illustrated in Figure 6.6.

Phase I Modeling Considerations (1979-1981)

Details of the West Side Interceptor river crossing are shown in Figure 6.7. The crossing is accomplished through a 42 in. pipe under the river, and 48 in. pipes near the end on each side of the river. Maximum head differential at the river crossing is based on the spillover weir height at the Weiss Street Station from the West Side interceptor. This head range can vary between negative values and 8 feet depending on the flow conditions in the East Side Interceptor at the junction.

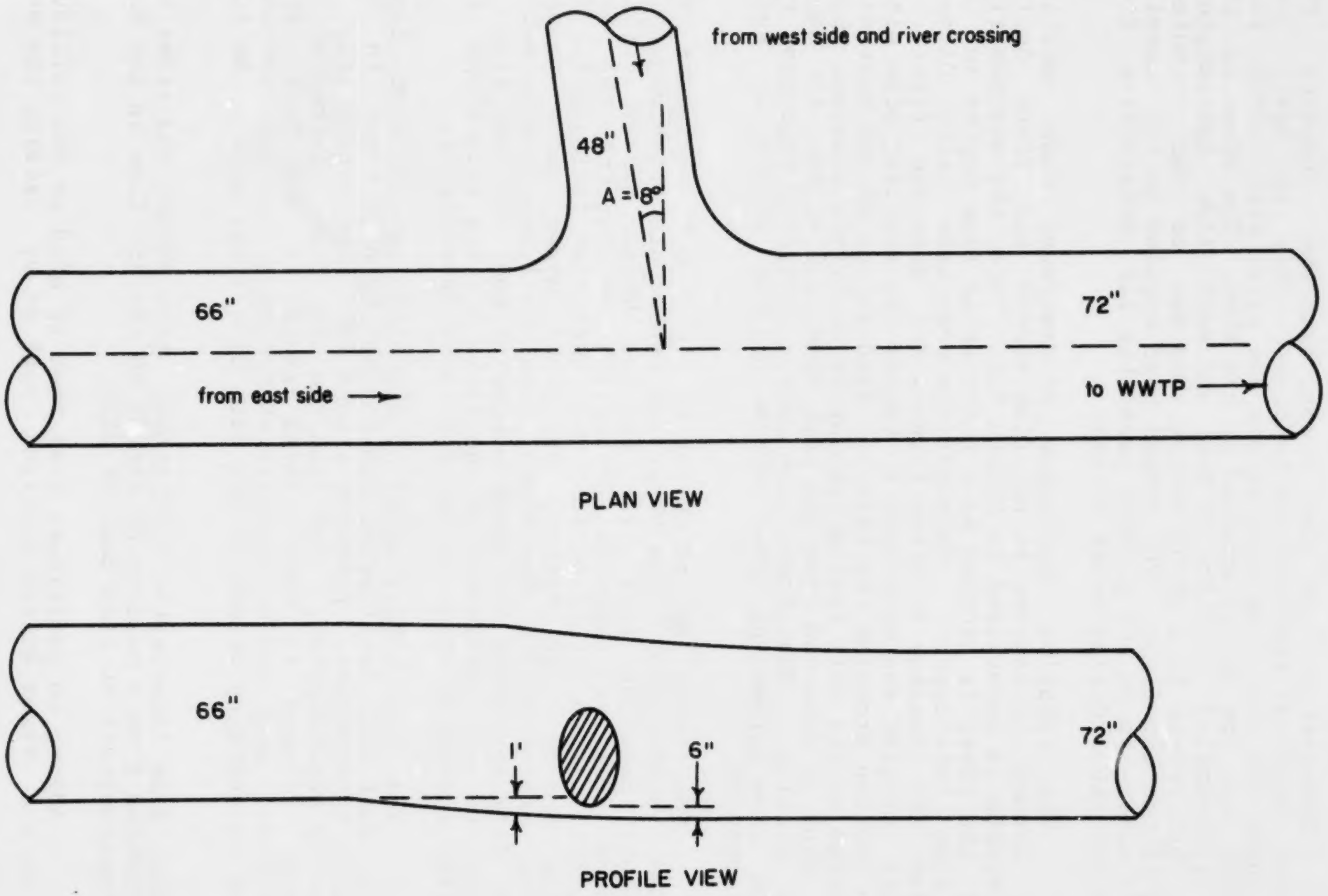

from west side and river crossing
48"
A = 8°
66"
72"
from east side
to WWTP
PLAN VIEW
66"
72"
1'
6"
PROFILE VIEW

Figure 6.7 Details of river crossing, Saginaw.

Examination of the river crossing suggests two important physical restrictions to flow. One, the small pipe sizes used to carry the flow across the river will limit flow depending on the energy applied to the flow. The other is the near right-angled junction of East and West Side Interceptors which may result in a large energy loss due to the turbulent mixing of the two flows. The actual head applied to the crossing was expected to be the primary parameter for determining flow across the river during storm events.

The relative importance of East/West Side junction energy losses in Saginaw to the flow across the river during storm events is summarized in Figure 6.8. Here, the maximum flow across the river is plotted as a function of flow depths in the East Side Interceptor. Calculations were made using Darcy's equation and average friction factors for concrete pipes. A computer program was developed to handle the changing pipe sizes in the section crossing the river. Head values on the West Side were based on the weir evaluations at Weiss Street Station, above which flow is diverted from the West Side Interceptor to Weiss Street Station. Therefore, the curves presented represent the maximum flow across the river just as overflow occurs at Weiss Street Station.

The lower set of curves in Figure 6.8 represent the pre BMP conditions. The upper set represents flows under an additional head of five feet at the Weiss Street Station (presumed relative easy to accomplish by raising the weir elevation).In each set of curves, the lower line included K = 1.8 junction energy loss factor, while the upper curve ignores junction energy loss. Therefore, the two lines in each set of curves form the upper and lower bounds of what the true flow is expected to be. Estimates of the ratio Q are also shown for different levels of stage in the East Side Interceptor.

The most likely range of operation is when high flows in the West Side Interceptor are matched by high flows in the East Side Interceptor, probably around full pipe conditions. As Figure 6.8 demonstrates, flow from the West Side Interceptor is severely reduced as water level rises in the East Side Interceptor, and can be entirely shut down if two and one-half feet of surcharging occurs in the East Side Interceptor. At full pipe East Side Interceptor conditions, flow across the river has been reduced from a maximum of around 60 cfs (no flow in the East Side Interceptor) to less than 36 cfs.

Adding an additional five feet of head on the spillover weir at the Weiss Street Station, such as by raising the weir

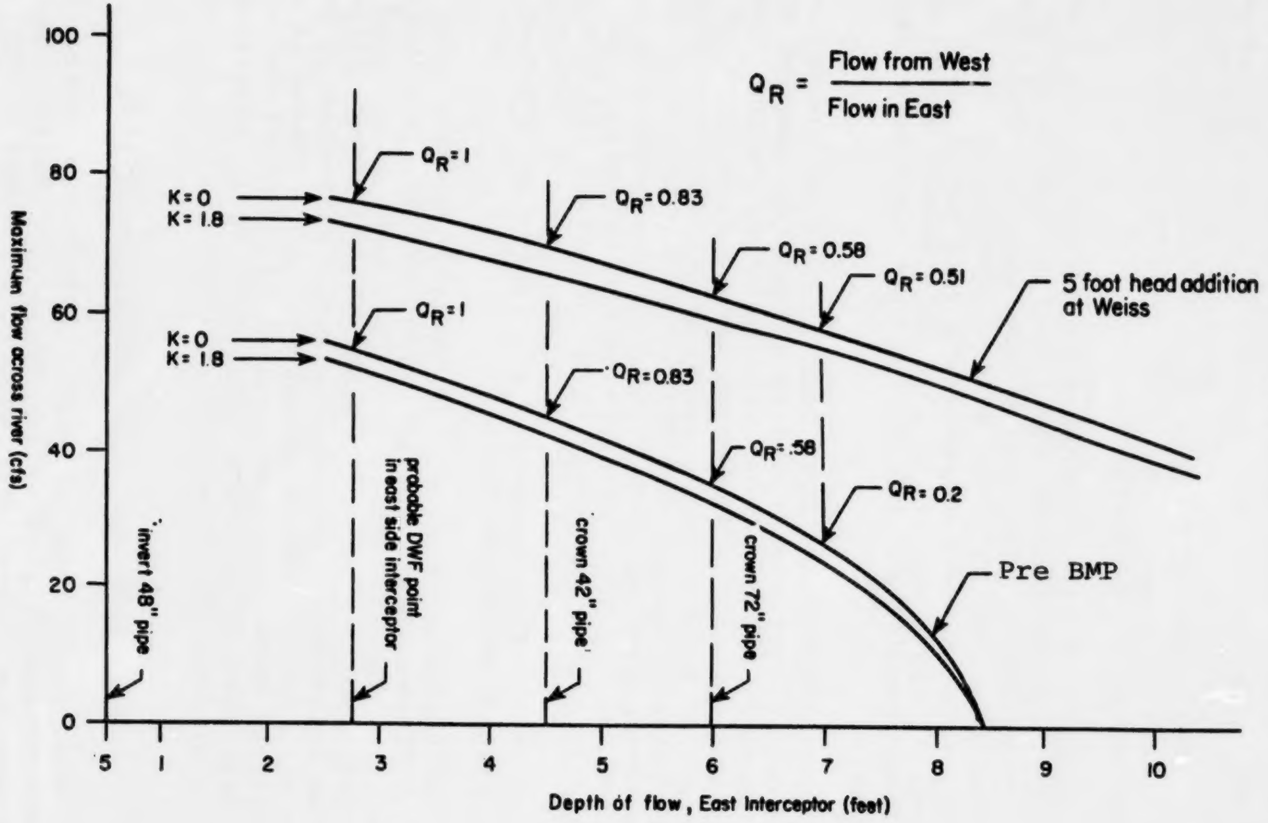


Figure 6.8 **Analysis of Saginaw River crossing discharge levels.**

elevation into the station's wet well, does a great deal to increase flow across the river. This is particularly true in the operating ranges of high East Side Interceptor flow. At full East Side pipe flow, flow across the river can be increased from below 36 cfs to above 60 cfs, and the gain is even greater for surcharged conditions in the East Interceptor. However, even for the more unlikely events of high West Side Interceptor flow and low East Side Interceptor flow, the gain by increasing the head differential is still substantial.

Figure 6.8 also demonstrates the relatively unimportant role played by energy loss at the junction itself. Gains made by improving head loss conditions at this junction could be no more than a few cfs, with these gains decreasing as East Side Interceptor flow rises and becoming very small in the most likely range of operation (QR = 3). This result suggests that the junction itself may not be the best candidate for future improvements.

The additional five feet of head at the Weiss Street Pump Station will not significantly impact the pumping pool level at the Hancock Street Pump Station considering level pool conditions but due to the increased water elevation (and the net head reduction between Weiss and Hancock) will serve to reduce flow rates between Hancock and Weiss. This result will reduce the pump chattering noticed from the Hancock records. It would be advantageous to further investigate and optimize the relative turn-on elevations of the pumps at Weiss Street and Hancock Street facilities to ensure initial operation at Hancock where storage/treatment is currently available as opposed to the untreated pumpage at Weiss Street.

In summary, theoretical analysis of the river crossing strongly indicates that flow is being severely restricted during storm events. This is due to the small pipe sizes used to make the crossing, and the lack of necessary high energy differentials to overcome friction in the small pipes, particularly when flows are also high in the East Side Interceptor. Based on empirical and theoretical work on energy loss at pipe junctions, the nature of the East/West Interceptor appears to be a relatively minor cause of river crossing flow restrictions.

Phase 3 Modeling Considerations (1984)

The above analysis was based on system hydraulic conditions as established during Phase 1. During the Phase 3 analysis it was determined that the influent pump wet well includes high and low levels alarms set at 551 and 556 feet, respectively. Original design and construction hydraulic profiles had indicated an operating wet well elevation of 547 ft. Actual operation at this elevation can cause severe pump cavitation.

The relative elevations of the WWTP influent pumps wet well and the 6 foot East Side Interceptor are such that normal plant operation between these alarm points backwaters the interceptor to varying degrees. Operation of the WWTP influent pumps is performed manually and higher or lower wet well levels are possible. Level pool backwatering at the alarm elevations translate to depths of 1.5 ft and 6.5 ft in the 6 ft diameter East Side Interceptor at the junction of the 48 inch West Side Crossover. Given that the 48 inch crossover line invert is 0.5 ft. above the 6 ft interceptor invert, backwater effects range from 1 ft depth to 6 ft depth (2 ft surcharge) in the crossover line.

The crossover analysis prepared in 1979-1981 (Phase 1) presented in Figure 6.8 was modified in Phase 3 considering interceptor wet well alarm levels and the results are depicted in Figure 6.9.

Low alarm (551 ft) and high alarm (556 ft) WWTP wet well induced backwater levels are indicated on Figure 6.9 at the interceptor depths of 1.5 and 6.5 ft respectively. In addition, a line indicating the hydraulic capacity of the interceptor with varying depth is shown. The significance of this line is that any wet well level and total interceptor flow condition falling to the left of the line will induce an interceptor liquid level governed by the interceptor flow rate and will be unaffected by backwatering. The actual interceptor depth will be as indicated by the intereceptor flow rate at the capacity line.

Wet well levels and interceptor flow rates to the right of the capacity line indicate backwater-controlled conditions. Flow in the East Side Interceptor and the West Side Crossover will be a function of the degree of surcharging in each line. Surcharging of the West Side Interceptor crossover is limited by spillover into the Weiss Street Station wet well.

Under conditions existing prior to the weir modifications implemented under the reduced BMP plan, crossover flow was limited to the rates indicated by the lower "pre BMP" line shown in Figure 6.9. Assuming a low alarm WWTP wet well level, the maximum crossover flow was about 53 cfs. Under potential high wet well conditions, (even during dry weather), crossover flow was limited to 30 cfs prior to spillover at the Weiss Street Station (assuming level pool conditions). Considering head losses between the junction and the WWTP, the maximum crossover rate would be 25 cfs. As indicated on Figure

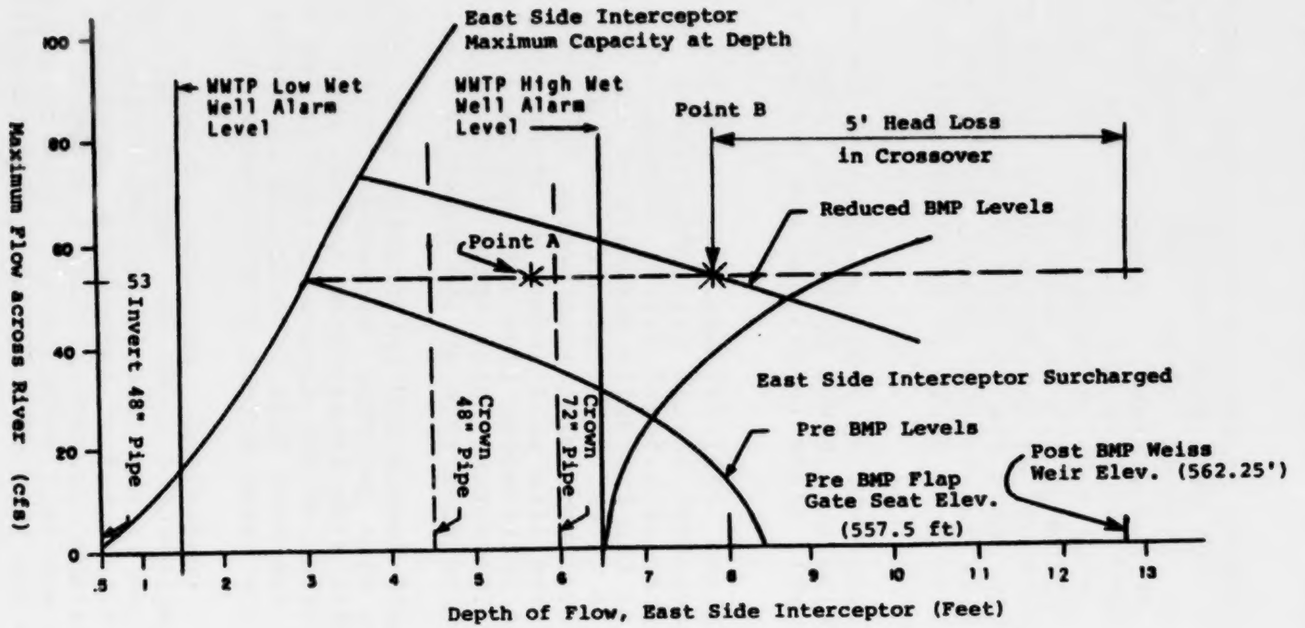


Figure 6.9 Verification of Weiss Street Pump Station Weir Modification Effects on Crossover Rates

6.9 the spillover elevation into the Weiss Street Station is 8 ft above the East Side Interceptor invert and only 1.75 ft above the WWTP wet well high alarm water level. Head losses within the East Side Interceptor and the crossover and occurences of wet well levels above normal may contribute to spillover into Weiss at minimal crossover rates. Modification of the Weiss Street wet well inlet by increasing the spillover elevation by 5 ft eliminated early spillover and maximized crossover flow.

Estimated potential crossover rates after reduced BMP modification are indicated by the upper line of Figure 6.9. The maximum crossover rate under optimum conditions (low alarm wet well level, no East Side Flow) is 73 cfs. The maximum crossover rate under high alarm wet well level conditions is estimated to be 50 cfs. Adjusting this value for surcharge due to head loss in the interceptor between the WWTP and the junction indicates a maximum crossover of 47 cfs.

In sum, under optimum conditions (low wet well, no upstream East Side flow) the maximum crossover rates would increase from 53 to 73 cfs due to the weir modifications. Under the condition of high wet well and no upstream East Side flow the maximum crossover rates would increase from 25 to 47 cfs due to program modifications. Upstream East Side flow will always exist and this flow will induce greater head loss and surcharging of the interceptor further reducing the potential crossover flow. The conclusions of this analysis are that modification of the Weiss Street Station spillover weir level will increase the crossover rate by about 20 cfs under all conditions and that operation of the WWTP wet well within the alarm range can vary the crossover flow rate by 25 cfs.

Post Reduced BMP River Crossing Evaluation

One aspect of the post evaluation (1984) was to document the improved river crossing flow regime by evaluation of the WWTP wet weather flow records. It was expected that greater volumes of flow would reach the plant during storm events after Weiss Street Pumping Station weir modifications were performed.

Following the Nov. 1, 1984 event high WWTP flow rates were observed for several days. Total flows were greatly above additional volumes expected due to Weiss Street weir modifications. Field investigations discovered a log jarring open a backwater gate allowing river water into the West Side Interceptor. Although this situation rendered invalid any attempt to document relative storm related flow volume increase at the plant, the several days of additional inflow provided a near perfect situation for evaluating river crossover flow conditions. During the period of river inflow, WWTP influent rates remained near 71 cfs for several days, and the West Side Interceptor level was reported to be just at the Weiss Street weir level. Based on the early morning WWTP flow rates prior to, during and after this event it was estimated that 37 cfs of river water entered the interceptor. Remaining flows were estimated as: other West Side - 16.6 cfs, upstream East Side - 9.8 cfs and additional downstream - 8 cfs based on the system distribution detailed in Chapter 5.

Figure 6.9 was utilized to demonstrate that this data verifies the predicted potential crossover rates. The only unknown factor was the WWTP wet well elevation. Estimating head loss in the interceptor between the junction and the WWTP for the 71 cfs flow rate as 4.2 ft and assuming the low alarm wet well condition, results in a depth of flow in the East Side Interceptor of 5.7 ft. Point "A" on Figure 6.9 indicates that for the estimated crossover rate of 53.6 cfs this data set falls to the left of the predicted line. Increasing the wet well liquid elevation above the low alarm level would shift the data point to the right closer to the prediction line. Alternatively, using the West Side Interceptor level and estimating the head loss of 53 cfs through the crossover at 5 ft placed the East Side Interceptor data point , Point "B", just on the prediction line. This would indicate a WWTP wet well level of 2.3 ft above the low end of the 5 ft alarm range, a quite probable condition.

Based on this analysis and the uncertainty of the exact wet well condition it appears that the prediction line accurately portrays maximum river crossing flow rates prior to spillover to the modified Weiss Street Station wet well.

6.7 Runoff Water Quality

Runoff water quality is calculated from three components: dry weather sewerage, cumulated dry weather sewerage deposition and street washoff.

The dry weather BOD and suspended solids (TSS) mass loadings at the WWTP were used as data inputs in the simulation model. Total dry weather mass loading at the WWTP was apportioned to the different catchment areas in the simulation model on the basis of relative contributory dry weather discharges. This procedure implies uniform dry weather flow concentrations throughout the system.

Since most of Saginaw is flat and dry weather sewerage deposition are expected to be considerable, pollutant deposition loadings during dry intervals between storm events were included in the simulation model. Deposition rates were based on the above calculated long term dry weather flow averages. Empirical equations developed by EDP (1) were used to approximate daily deposition rates for the more important drainage areas. These equations require as input the following quantities: a) total collection system pipe length; b) average collection system pipe slope; and c) the average per capita waste rates. For simulation work a constant daily deposition rate equalling 15% of the dry weather pollutant loading for each catchment area was used throughout the system. Comparison between the empirical results derived for four catchment areas and an arbitrary assumption of 15% of the total dry weather load is shown in Table 6.3 and suggest that the 15% simplification is reasonable. Resuspended deposited loadings are assumed to be distributed evenly throughout any rain events. It is further assumed that all deposited loadings would be scoured and completely flushed in an given event, that is, there would be no carryover of residual loadings to the next event. This assumption may distort the "first flush" estimates for both high and slight runoff events, but would yield on the average, satisfactory estimates.

Street runoff water quality was estimated using empirical equations based on rainfall and percent imperviousness of each catchment area (2). The empirical equations given below for street runoff were adjusted to better match the expected values in Saginaw. The equations used as a starting point in the calibration analysis are as follows:

$$\text{TSS (lb/acre)} = I/(0.136 + 0.00384\ (I)(\text{rain})^{1.05}\ )$$

$$\text{BOD (lb/acre)} = (614/I)(\text{rain})0.82 \qquad (2)$$

where I is percent imperviousness and rain is the total for the event in inches.

Because street washoff water quality is the most difficult component of the analysis to predict without adequate local data, manipulation of the empirical curves developed in other communities was considered necessary. Dry weather flow loadings and deposition rates were left unaltered. Initial simulation runs showed both excessively high TSS and BOD loadings and runoff concentrations. Street washoff pollutant formulations were altered (reduced) to better match Saginaw conditions.

Several of these curves for a number of different municipalities are shown in Figure 6.10. Imperviousness is an implicit parameter in developing these curves. Saginaw was considered to be cleaner than the municipalities used to derive the equations (streets are well swept, for example) and it was expected that Saginaw washoff solids loadings to be on the lower end of the group. Saginaw loadings fell to the higher end of the group of curves shown in Figure 6.10 using a 34% overall average

TABLE 6.3

COMPARISON OF COLLECTION SYSTEM DAILY DEPOSITION LOADINGS

Collection System	Method A		Method B	
	TSS*	BOD*	TSS*	BOD*
ADAMS	530	300	540	240
WEISS	1100	600	1700	720
WEBBER	520	300	960	420
McCOSKRY	280	160	440	200

Method A: Daily Dry Weather Collection System Deposition Loadings Equal to 15% of Total Daily Load.

Method B: Prepared using empirical equations (1)

* - pounds solids deposition per day

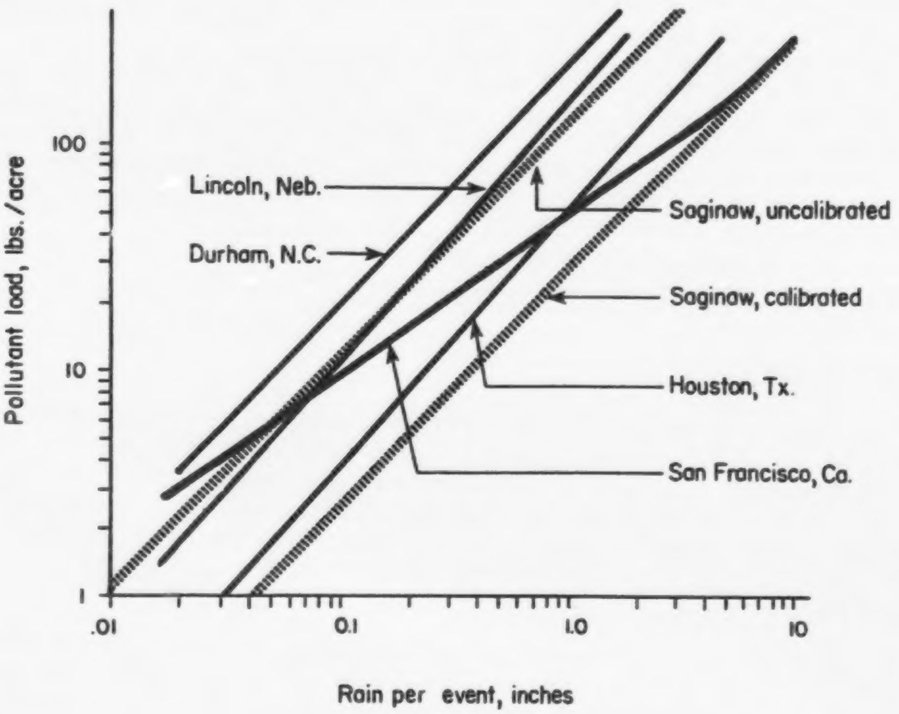


Figure 6.10 Saginaw suspended solids generating equation compared to other municipalities.

imperviousness. The solids washoff formulation was reduced by a factor of four (See Figure 6.10) to lower the solids curve for Saginaw in comparison to other municpalities, and to better match the available Saginaw suspended solids data.

The street washoff BOD loading formulation given by equation was also adjusted. For example, the computed ratio of TSS to BOD loadings using the equation (2) is 6:1 for a 0.1 inch storm. More common ratios are 20:1, 24:1 and 7:1 for residential, industrial, and commercial areas, respectively (3). This comparison suggested the BOD predicting equations used for simulation were also high. EDP used data from previous work (4) to adjust the BOD predicting equations. Literature surveys of BOD street accumulation rates had found these rates to be equal to 0.15, 0.21, 0.64, 0.53 and 0.11 pounds per acre per day for single family, multiple family, commercial, industrial, open land uses, respectively. An average of 0.3 pounds per acre per day was estimated for Saginaw land use mix. Using an average of four days between rain events and a nominal washoff factor, 0.1 inch rain will produce approximately 0.3 pounds per acre of BOD. This value is one sixth the value from the simulation equations. The equations were therefore lowered by a factor of six.

6.8 WWTP Simulation

The TSS and BOD efficiencies at the WWTP were established at 93% and 89% using long-term average data. Maximal plant throughput flow of 128 cfs was allowed. Additional flow above 108 cfs was assumed to be split at the grit chamber with half bypassing the main plant facility. Excess flow above 128 cfs was allowed to backup resulting in overflow at the Weiss Street Pump Station.

Phase 1 modeling investigations (1979 -1981) indicated a potential for increased utilization of the WWTP during wet weather. It was envisioned that increased utilization of the WWTP capacity during wet weather would be an extremely cost-effective option in the overall combined sewer control management approach for Saginaw.

During the Phase 1 modeling, it was believed that additional wet weather flow could be handled. A rudimentary analysis was conducted to assess the pollutant removal effectiveness at the WWTP during wet weather flow conditions. BOD removal efficiencies were investigated during high flows and compared to low flow conditions to ascertain the feasibility of this approach. A year and a half period of WWTP daily operating records for 1978-1979 was examined. The BOD removal for the WWTP averaged approximately 89% for both wet days and dry days. It was tentatively concluded that the WWTP could handle higher wet weather flows without compromising the plant's pollutant removal capabilities.

The WWTP portion of the simulation model used for assessment of the reduced BMP and complete BMP program effectiveness is described as follows. Interceptor discharges beyond a pre-set maximum of 128 cfs was assumed to backwater and overflow upstream. All flows less than 108 cfs received full treatment. Discharges beyond 108 cfs but limited to 128 cfs were flow-split at the grit chamber with half the excess continuing through the plant and the residual by passed (and disinfected). All flows continuing past the grit chamber receive full treatment (BOD efficiency = 89% and suspended solids removal = 93%).

These assumptions approximate current plant operations. For small to moderate intensity storms the model assumptions very closely replicate practice. For major storms of short intensity, the assumed procedure is adequate provided the storage volume of the two aeration basins used for detention are not exceeded. The model slightly overestimates treatment efficiency for storm events in which the aeration tank volume is exceeded and through flow conditions result for the aeration tanks.

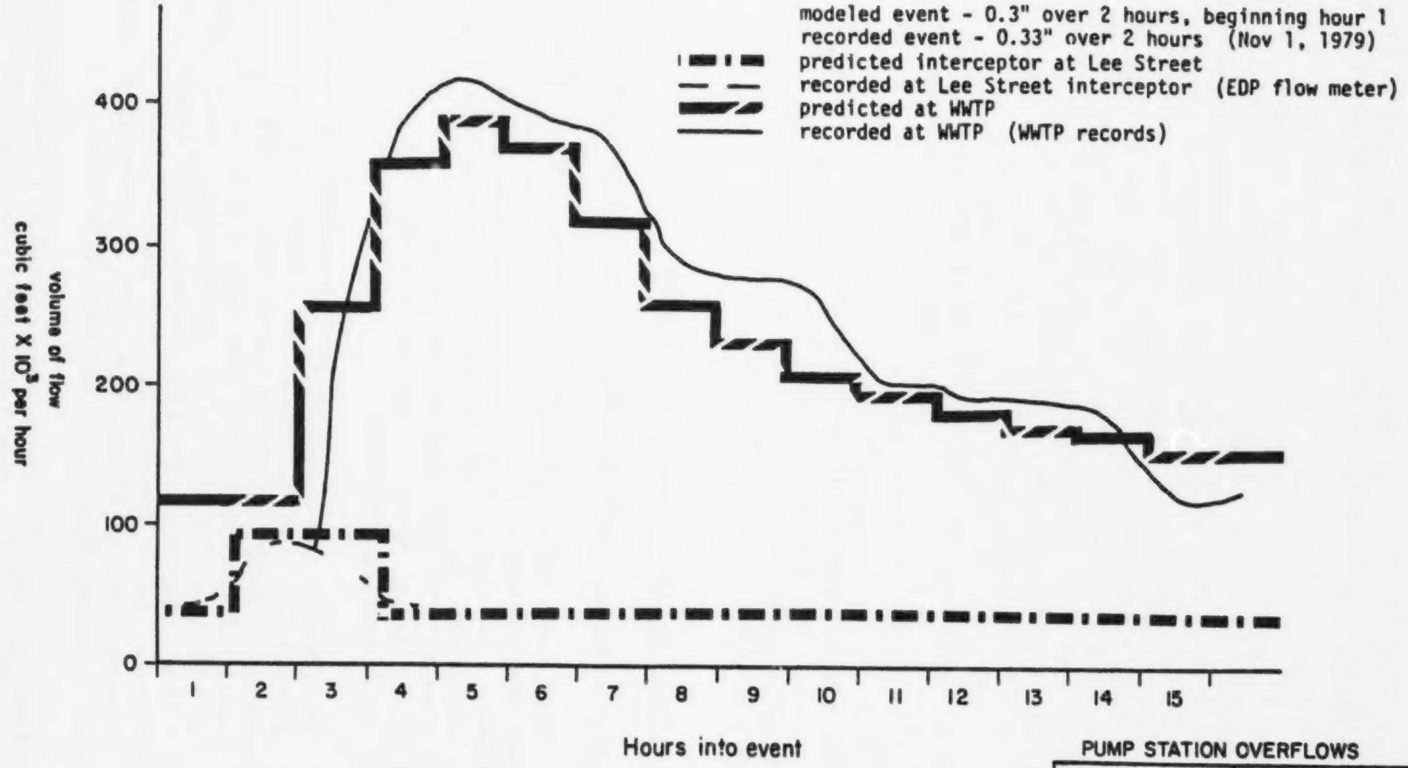
6.9 Model Calibration/Verification Efforts

Phase 1 Model Calibration - 1979 Flow Data

The Saginaw CSO simulation model was designed to provide a rough approximation of the degree and distribution of overflow problems. As such, and recognizing the severe limits of the available recorded calibration data, it was not appropriate to fine-tune the model to the degree model calibration may imply. Rather, available data were used primarily as a check for reasonableness, and actual calibration was kept to a minimum.

The hydraulic simulation as developed on the basis of the systems physical characteristics was not substantially altered after initial comparison to recorded values during Phase 1. The major point of concern was the operation of the regulator chambers as operation data were available only for four regulators in the Hancock Street area. Different regulator operational schemes were tested in the simulations analysis in selecting the scheme that yielded the most reasonable comparison of the predicted interceptor to measured flow data.

A number of single rain events were simulated to check the adequacy of the model. The reasonableness of the simulation model is demonstrated in Figure 6.11 showing the comparison



PUMP STATION OVERFLOWS

	Hancock	Weiss
predicted cubic feet X 10^3	475	630
from pumping plant analysis curves	400	680

Figure 6.11 Single event comparison of modeled and recorded events.

between a 0.3 inch modeled event occurring over a two hour period, and a 0.33 inch actual rain event occurring over two hours on November 1, 1979. Flow at the WWTP compared favorably with predicted flows peaking slightly less than recorded values. Flows at Lee Street on the West Side also compared well. Spot measurments (not shown) made on the East Side also compared well to predicted values. As shown in the table in Figure 6.11 the predicted pumped volumes from the Hancock and Weiss Street Stations closely approximate volumes computed from pumping records. For the same rainfall event the predicted temporal sequencing of flows at the Hancock Station, the Weiss Station and at the WWTP are shown in Figure 6.12 and agree with observed operation of the system.

Predicted overflows at the pumping stations compared well with recorded values. Figures 6.13 and 6.14 show the comparison of predicted and recorded stormwater pumped per rain event for the Weiss and Hancock Street Stations respectively. The circled data points represent recorded values developed in the pumping station analysis and the square data points represent predicted values for different rainfall events. The model somewhat underestimates spring conditions of high snowmelt and concurrent rainfall. Simulated overflows at the Emerson Street Pump Station occurred only for extreme rain events in accordance with the observed operation of the Emerson Street Station.

The results of a full year of simulations (1978) are compared in Table 6.4 with WWTP records that were available for the same period. The simulation model summarizes flow and water quality as rain contributions above a nominal dry weather loading. This calculation was performed so events of different durations could be compared with their total loadings. An attempt was made to discern the same statistic from the recorded WWTP records. Plant flow records were analyzed to eliminate a period around a rain occurrence. The estimation of wet weather flow contribution changes substantially depending on the length of period selected around a rain event. Table 6.4 demonstrates the range using two to four days as the period eliminated after a rain event to average the dry weather loadings. The simulated rain contribution fell within the range for excess flow at the WWTP.

Phase 1 - Calibration 1980 Water Quality

A storm generating combined sewer overflows was sampled on July 8, 1980. The sampled storm was 0.83 inches of rainfall occuring over a little less than two hours as recorded at the Saginaw WWTP. For comparison EDP simulated a 0.8 inch storm occuring over two hours.

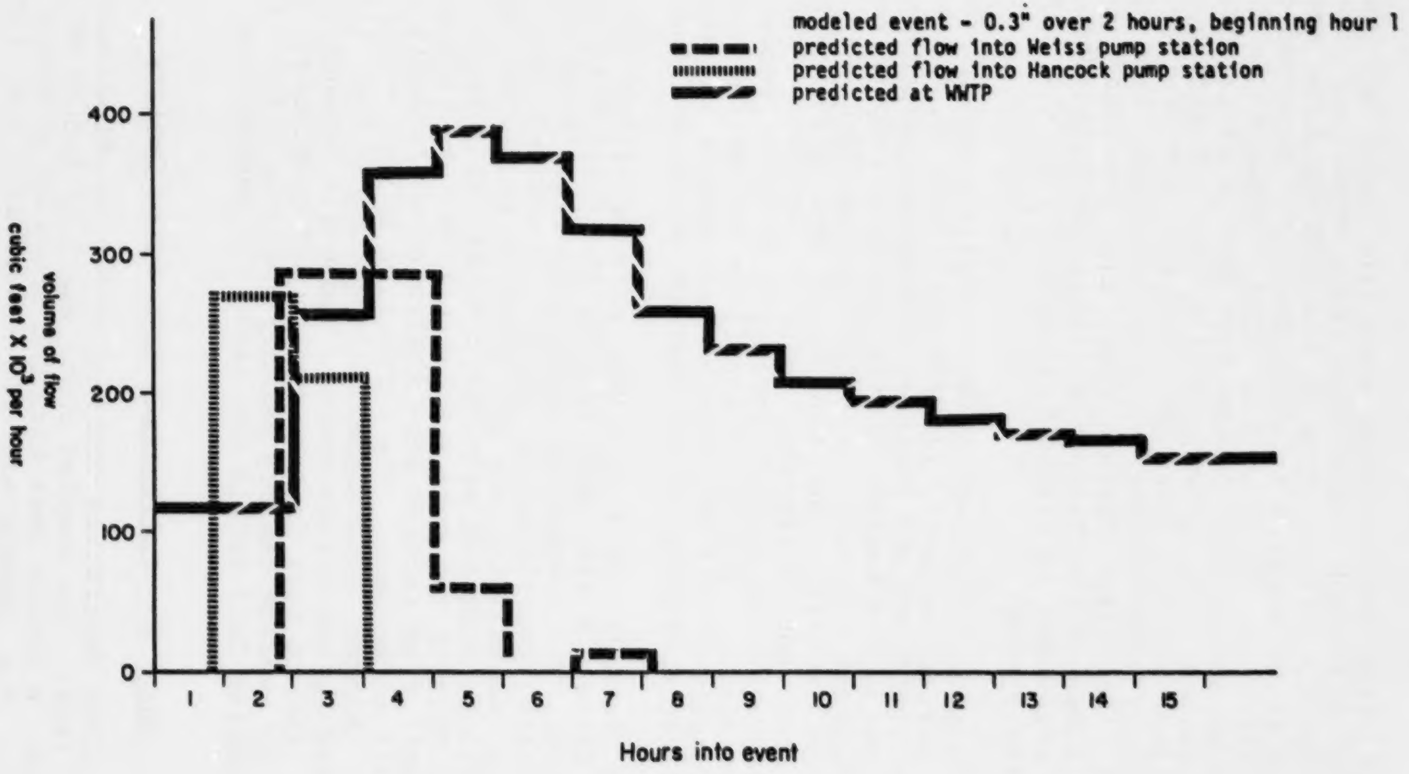


Figure 6.12 Single event time sequencing.

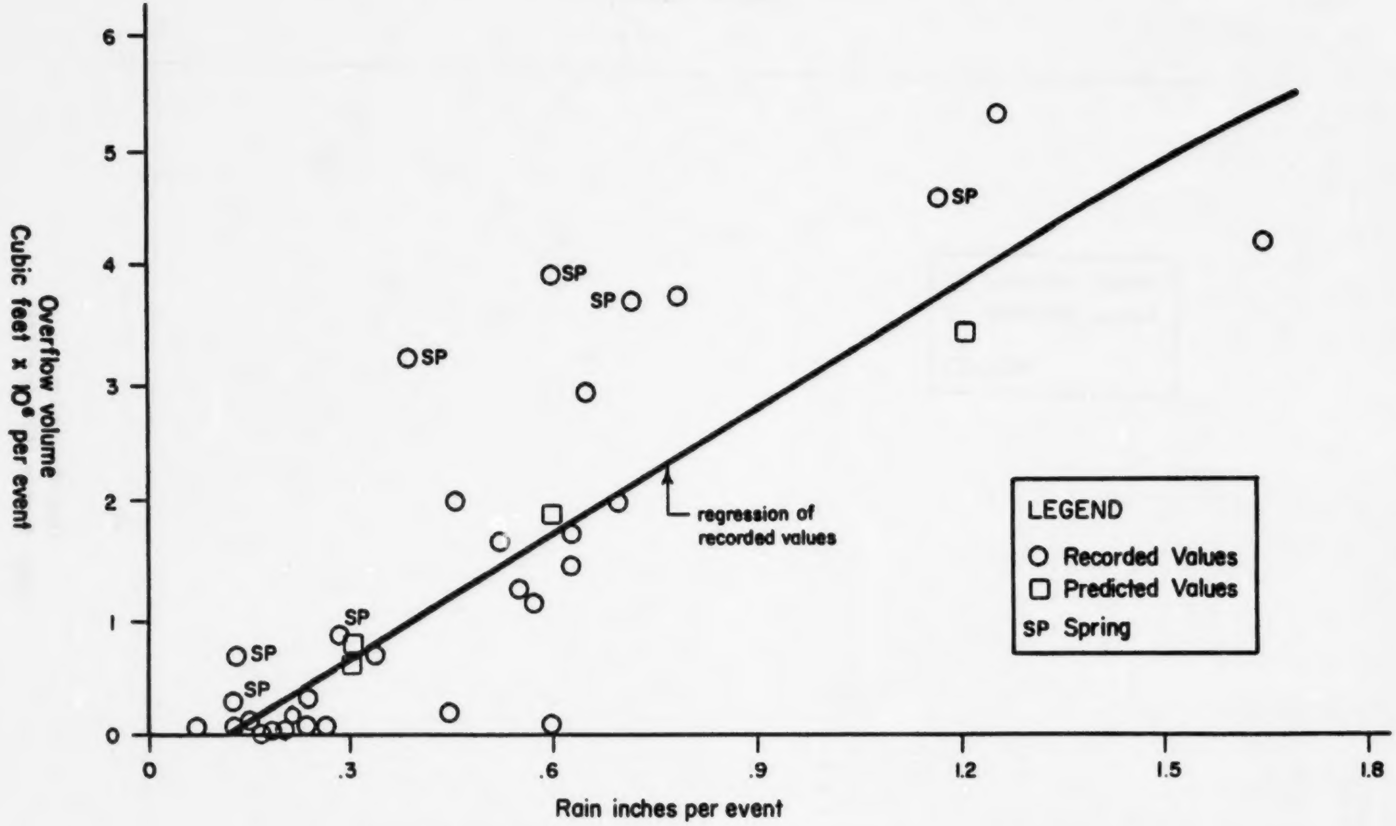


Figure 6.13 Overflow into Weiss Street pumping station 1978-1979.

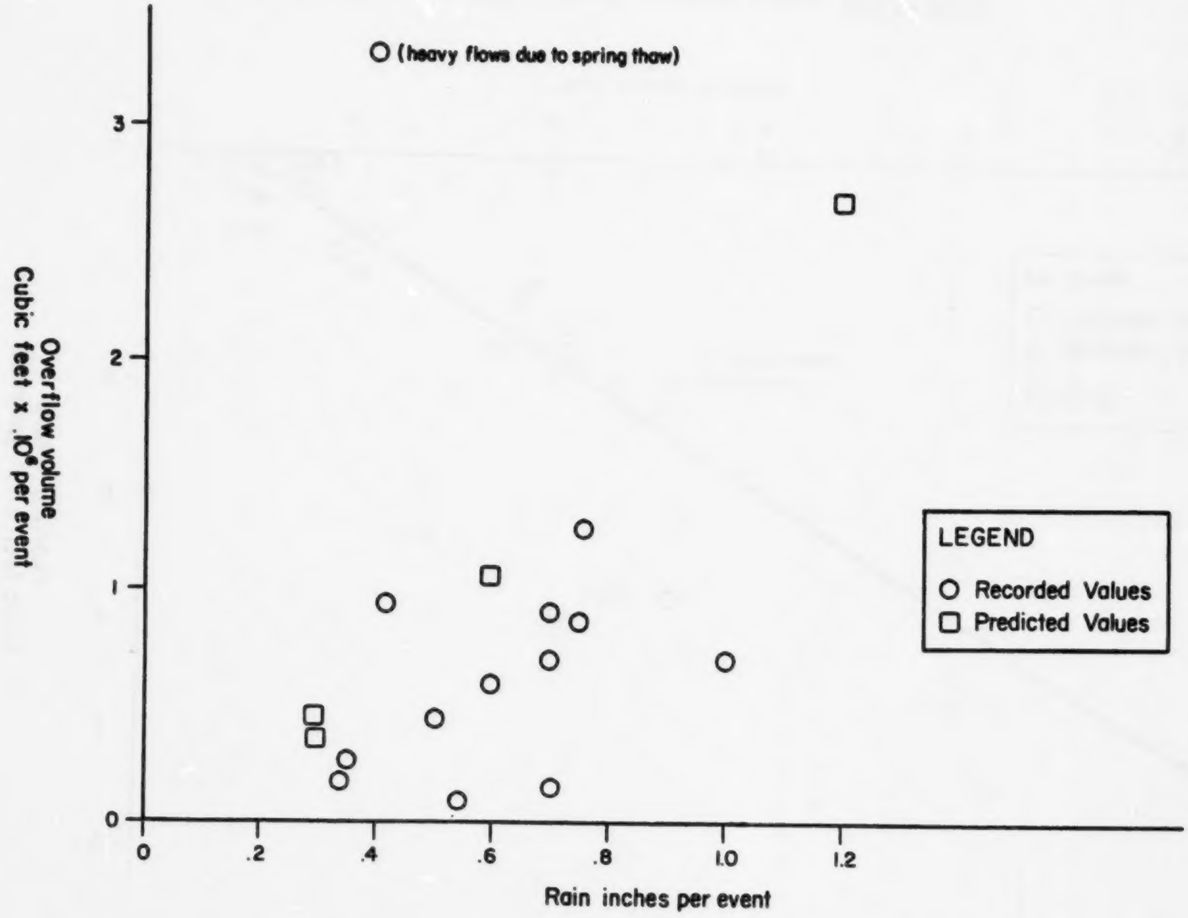


Figure 6.14 Overflow into Hancock Street pumping station 1978-1979.

TABLE 6.4

PREDICTED VERSUS RECORDED VALUES AT WWTP (1978)*.

	Predicted	Recorded
Total Volume of Flow x 10^6 ft^3	1071	1130
Volume of Rain Contribution x 10^6 ft^3	89	57-140 (estimated)
TSS x 10^6 lbs.	11.7	10.3
TSS Rain Contribution x 10^6 lbs.	1.9	0.8-1.2 (estimated)
BOD x 10^6 lbs.	6.2	5.7
BOD Rain Contribution x 10^6 lbs.	0.7	0.1-0.3 (estimated)

* - (1978: 304 day period, January - October.)

The EDP simulated storm generated slightly more overflow volumes and slightly lower flows at the WWTP. Not all overflow points were measured by the sampling program. The primary overflow point at the Weiss Street Pump Station was measured and compared well with the simulated value. Table 6.5 summarizes the hydraulic comparisons.

Measured water quality data was converted to total pollutant loadings in pounds for comparison to EDP simulated values (Table 6.5) Concentrations of pollutants were sampled, however, flows at the time of sampling were not available. Since the Weiss Street Pump Station uses constant capacity pumps, and for a storm of .8 inch only one 100 cfs pump would be expected on at any given time, an identical flow for each sample was assumed. Total pollutant loads were calculated using the numeric average of the water quality samples and the times the total volume pumped. The resulting total loads compare well with EDP simulated loads.

Phase 2 - Model Calibration

EDP directed detailed wet weather field investigations peformed by City of Saginaw personnel at several regulators during the summer of 1983 and an intense program between July and November, 1984. Completion of the reduced BMP construction program occurred at the end of September 1984 so that some events represent pre BMP and others represent post reduced BMP conditions.

As a check to the accuracy of the model developed in Phase 1, the hourly rain data for each storm event occurring during the 1984 pre reduced BMP period was used as a model simulation input. Modeled results included WWTP flow volumes at 40 to 65 per cent of measured volumes, Weiss Pump Station volumes at 40 to 125 percent of measured volumes and Hancock Pump Station volumes at 160 to 280 percent of measured volumes. Based on the consistently low values at the WWTP and consistently high values at the Hancock Street Pump Station the model was reviewed and modified in an attempt to improve field and model data correlation.

It should be noted that several changes to the Saginaw sewerage system had occurred since the original Phase 1 model calibration, the most significant of which was the alteration of relative initiation points at the Weiss Street and Hancock Street Pump Stations. This alteration was performed by the City to better utilize the storage capacity at the Hancock Street detention facility prior to pumped overflow discharge at Weiss Street and constituted a major change in system operation.

TABLE 6.5
MODEL CALIBRATION RESULTS - STORM EVENT, 7/8/80
FLOW/VOLUME RESULTS

A. Sampled Storm: 0.83 Inches July 8, 1980
Simulated Storm: 0.8 Inches

Overflow Volumes (10^6 cubic feet)

	Measured	Modeled
Weiss Pump Station	2.2	2.4
Hancock Basin	0.3	0.6
Fourteenth Street	*	0.9

* pump station was sampled gravity overflow was not so total overflow volume is not known.

WWTP Flows (mgd)

July 8, 1980	Simulated
45	37

B. Weiss Pump Station Overflow Water Quality Calibration of July 8, 1980

Measurements

Time (am)	BOD_5 mg/l	TSS mg/l
1:25	102	188
1:45	84	192
2:05	158	1032
2.25	102	656
2.45	74	46
	---	---
numerical average	104	423

total volume overflow = 16 million gallons

Total Pollutant Loadings at Weiss(pounds)

	Measurement	Simulated by Model
TSS	56,500	59,300
BOD_5	13,900	15,300

The model was re-calibrated based on improved knowledge of actual regulator operation, rainfall/runoff characteristics and intuitive estimation of system hydraulics. For this purpose the storm event occurring on July 10-11, 1984 was selected as it included a high rain accumulation (1.14 inches) and occurred prior to implementation of any elements of the reduced BMP Plan. Model modification in general included, varying regulator underflow/backwater gate characteristics, refining estimated times of concentration in several drainage subareas and alteration of interceptor stage/discharge dynamics.

The effects of these modifications, particularly of increased time of concentration, were such that event runoff durations increased and peak flow rates decreased reducing modeled pumpage volumes at the Hancock Pump Station and providing additional flow to the WWTP. For the 7/10-11/84 event final ratios of field data volumes to modeled data volumes were 1.15 at the WWTP, 1.05 at Weiss Street Pump Station and 0.90 at the Hancock Street Pump Station. It should be noted that for each event, rain data was recorded hourly at the Emerson Street Station and daily by NOAA at the City Water Works Building. Variances in rain data values at the two nearby locations, which represent variable rain conditions within the City, were typically much greater than the differences in field and model data recorded for the 7/10-11/84 and most other events.

Considering the close correlation of the 7/10-11/84 field and model data, the model was considered suitably calibrated for prediction of pre-BMP condition system flow volumes. Each of the 1984 monitored storm events occurring prior to completion of the reduced BMP plan construction was rerun using the upgraded simulation model. Results including field measured data to modeled data, volumes ratios and rain characteristics data are included in Table 6.6.

Model results are closely correlated to field measurements at the WWTP for the first 5 events. For the final event the field to model ratio of 1.52 is high based on Emerson Street Station rain data, however rain data also indicates the NOAA Station recorded 1.95 times the Emerson Street Station rain accumulation. In this case the average rain at both stations is 1.47 times the Emerson Street value and would probably produce model data closer to the field value.

Weiss Street Pump Station data tends to differ substantially from field data after the first event. Later events with high field to model volume ratios are probably due to the occurrence of these events during the construction period. A number of the original regulating devices at the regulator chambers were removed and a number of weeks passed before new regulating devices were installed. This would produce greater flows to the West Side Interceptor which in turn would require greater pumpage at the Weiss Street Station (prior to the station weir modification). High values for the last event correlate with the previous WWTP flow discussion.

As depicted in Table 6.6, the Hancock Street Pump Station data indicate reasonable correlation. Typically, the model predicts higher values than the field data which is probably due to the model assumption of a 300 cfs station capacity when potentially only 1 or 2 pumps actually operated simultaneously. Exact "turn on" water elevations for the three pumps are unknown and future study is required to optimize these locations. Data presented is overflow to the river at Hancock and does not include wet weather flows pumped to the detention basin and bled back to the interceptor. For this reason events indicating non-zero model data flow and zero from the field may be misleading since the event may have actually activated the Hancock Station but produced no flow to the river. In sum, variances between Hancock Station field and model data for the three locations are minor on an actual volume basis.

Phase 3 - Post Construction Model Verification

Monitored Storm events occuring in October and November of 1984 provided field data representative of field conditions existing after complete implementation of the reduced BMP Plan. All West Side regulating devices were replaced with vortex valve control regulators, one upstream storage chamber was completed and the Weiss Street Pump Station weir was modified. Unfortunately, few major events occurred during this period and for several of these events, field data validity required intuitive interpretation. Results are included in Table 6.7.

An event on 10/18/84 indicated reasonable agreement between field and modeled data at the WWTP. The Weiss Street data indicated a low model value which proved to be a consistent trend for all events as discussed below. No field data was recorded at the Hancock Street Facility and the model data was combined with the 10/21/84 value for evaluation.

During the 10/21/84 event, one regulator flapgate became lodged-open allowing river water to flow into the interceptor until manually corrected. Data at the WWTP indicates good correlation between model data and first day WWTP recorded flow. Thereafter, plant flows continue to remain above normal, presumably due to the open gate since similar rain conditions during other events produced only single day effects at the WWTP. Weiss Street model data was again typcially low. Model data for

TABLE 6.6

PRE REDUCED BMP CONSTRUCTION
TOTAL FLOW - 1000 CU FT

	TREATMENT	PLANT		WEISS PUMP STATION			HANCOCK PUMP STATION		
DATE	FIELD	MODEL	R	FIELD	MODEL	R	FIELD	MODEL	R
7/10-11	5300	4594	1.15	3360	3212	1.05	1207	1339	0.90
8/5	2620	2432	1.08	613	791	0.77	0	518	0
8/8	3120	2995	1.04	3032	1837	1.65	220	309	0.71
8/18	2500	2233	1.12	1950	1160	1.68	0	112	0
8/27-30	7715	6900	1.18	3233	4320	0.75	-	2626	-
9/7-11	5591	3671	1.52	2452	1533	1.60	0	0	1

R = Ratio of Field/Model

the 10/21/84 event at the Hancock Street Station when combined with the 10/18/84 event model data, agreed well with the recorded value representing both events. Correlation between field and model data for this event is considered good.

Following a minor rain on 10/25/84, WWTP flow remained high for several days producing flow equal to the model value after 2 days and 35 percent greater after the third day. The NOAA Station recorded twice the rain recorded at the Emerson Street Station (used in the model) for the three day period which again indicates the significance of storm intensity variation throughout the drainage area. The other significant aspect of the event is that minimal flows were modeled and also recorded at Weiss Street and Hancock Street facilities somewhat verifying the "low-end" calibration of the model.

Both events occurring during November in Table 6.7 were subject to adverse conditions. On November 1 the large Weiss Street flapgate lodged open, allowing large amounts of river water to enter the system. On November 8-11 a four day wet period was mostly snow, making accumulations and runoff predictions difficult. In general WWTP and Hancock Street Station flows modeled were in the range of recorded flow and Weiss Street Station flow volumes predicted in comparison to measured values were low.

Low predicted flow values at the Weiss Street Pump Station are probably attributable to reduced river crossover flows, a direct result of the wet well levels at the WWTP being manually controlled.

As part of this study's recommendations it is envisioned that the pump control systems at Hancock Street, at Weiss Street and the WWTP be evaluated and optimized for maximum pollution reduction using existing facilities. The general conclusion resulting from the foregoing discussion is that based on currently available field data, the model, as modified to match pre-construction data, reasonably predicts the effects of the reduced BMP implementation as recorded at the WWTP and at the Hancok Street Pump Station.

6.10 Phosphorous Modeling Details

Phase 1 & Phase 2 Considerations (1979-1983)

In addition to the TSS and BOD parameters previously detailed, an analysis of potential total phosphorous removals were performed for each simulation. Evaluation of phosphorous loads in wet weather and dry weather flows and potential removals

TABLE 6.7

POST REDUCED BMP CONSTRUCTION STORM DATA
TOTAL FLOW - 1000 CU FT

	TREATMENT PLANT			WEISS PUMP STATION			HANCOCK PUMP STATION			Rain Accumulation		
DATE	FIELD	MODEL	R	FIELD	MODEL	R	FIELD	MODEL	R	Emerson	NOAA	R
10/18	3475	2889	1.2	1768	849	2.1	1136*	1188	0.96	0.75	0.67	1.18
10/21	4679	4862	0.98	4000**	1439	2.8				0.79	1.0	1.27
Day 2	68182		1.4									
10/25	935	2121	0.44	108	0	-	0	0	1	0.17	0.35	2.06
Day 2	2139		1.0									
Day 3	2807		1.32									
11/1	2807	3379	0.83	2562	839	3.0	774	376	2.0	0.61	0.80	1.31
Day 2	60162		1.78									
11/9	3060	3333			605			0		0.73	0.65***	-
Day 2	6444											
Day 3	10900	4367		2454	387		540	0	-			
Day 4	11400	7700	1.48		992	2.47						

* - Combined Data 10/18 & 10/21

** - Backwater Gate Reported Stuck Open

*** - Rain/Snow Mixture

at the WWTP, began with a review of existing phosphorous concentration records at various locations in the system.

A pilot study performed between October and December 1969 for assessing various methods of phosphorous removal reported WWTP, raw sewerage total phosphorous concentrations to range from 3 to 10 mg/l and averaged 6.4 mg/l (5). Review of report data indicated wet weather flow concentrations to range from 3 to 5.3 mg/l. During the July 8, 1980 storm event samples were collected at selected overflow points. Total phosphorous concentrations ranged from 0.2 to 7.5 mg/l at the Weiss Street overflow and from 0.15 to 1.3 mg/l at the Fourteenth Street overflow. Flow rates were not recorded and flow-weighted average concentrations could not be computed. Total phosphorous concentrations of several influent samples at the Hancock Storage/Treatment Facility were averaged to yield a concentration of 2.8 mg/l.

Table 6.8 presents summary results of total phosphorous loadings to the WWTP for the period from October, 1978 to September 1979, as determined from treatment plant records. Total phosphorous concentrations were highest during fall months presumably due to the effects of decayed foliage. Average WWTP influent phosphorous concentrations for all weather conditions are somewhat less than reported for the 1969 October to December pilot study. Wet weather concentrations during October and November, 2.5 and 3.46 mg/l, respectively, are comparable to the 1969 results. It is assumed that the dry weather concentration reduction was due to a change in consumer use of phosphorous-based products during the time between the two data periods and that the more recent data accurately represent existing conditions.

Average monthly WWTP total phosphorous removals for the 1978-79 period varied from 63.5% to 82.2 % and averaged 75%. Average monthly wet weather concentrations indicate a yearly average concentration of 2 mg/l. Further analysis of WWTP removal efficiencies during wet weather conditions indicated that the WWTP could adequately pass increased flow without efficiency reduction. For the simulation model calculations, total phosphorous entering the WWTP during wet weather flow was set at a nominal concentration of 2 mg/l and a WWTP removal was set at 75 percent during Phase 1 and Phase 2 evaluations. Based on the range of total phosphorous concentrations observed at Weiss Street, Fourteenth Street, and at the Hancock Facility, a total phosphorous concentration of 2.8 mg/l was set as representative of all overflows. Considering the floatable nature of materials within CSO and the post storm draindown of late event cleaner stormwater to the WWTP it would be expected that overflow concentrations at regulator chambers would be

TABLE 6.8

WASTE WATER TREATMENT PLANT

TOTAL PHOSPHOROUS SUMMARY

Month	Average Influent Concentration MG/L	Average EFFLUENT Concentration MG/L	Average Removals %	Wet Weather Average Influent Concentration MG/L
7/79	2.3	.99	63.8	1.74
8/79	2.5	.89	63.5	1.98
9/79	3.0	.94	68.9	-
6/79	2.86	.85	68.6	2.45
5/79	2.6	.89	65.8	2.08
4/79	2.2	.76	66.2	2.1
3/79	2.2	.59	72.8	1.52
2/79	3.9	.67	82.2	-
1/79	3.5	.86	75.4	-
12/78	3.5	.85	75.2	-
11/78	3.3	.88	71.2	2.5
10/78	3.4	1.06	76.3	3.46
			75.	2.00

greater than WWTP concentrations. These concepts are reflected in the above values.

Phase 3 Additional Considerations (1984)

The above phosphorous analysis was based on data collected during the late 1970's when standard practice at the WWTP included the use of waste pickling liquor or ferrous sulfate for phosphorous removal. In the last several years, lower influent phosphorous concentrations have enabled the City of Saginaw to meet effluent standards without the continuous use of this specialized phosphorous treatment. The effects of discontinued pickling liquor treatment on overall phosphorous removal rates were investigated in Phase 3 of this study.

During this period of the study, U.S. EPA requested EDP to investigate the soluble fraction of the total phosphorous load as it is the soluble fraction which is most readily available to aquatic growth in Saginaw Bay.

Up until this time, the standard procedure at the WWTP included evaluation of total phosphorous concentrations. Plant records for summer/fall periods in the years 1978 and 1979, the base data years of the original study, were available and were used in conjunction with current plant data for analysis of phosphorous removal efficiencies. Historical data review was limited to total phosphorous removal efficiency determination. Soluble phosphorous data was attained as a special task during this period.

The Phase 3 (1984) field investigation indicated lower WWTP influent total phosphorous concentrations than reported during earlier periods. Average WWTP concentrations for all flows for the 5 month June to October period were 1.55 mg/l. Wet weather samples at the WWTP grit chamber indicated a weighted average of 1.3 mg/l, although "first flush" values were as high as 6 mg/l. Wet weather concentration in the Weiss Street Pump Station averaged 1.74 mg/l.

All loading calculations performed in the evaluations of Chapter 10 of this report were made using values of 1.3 mg/l for wet weather flows to the WWTP and 1.74 mg/l for system overflows. Previous model analyses were based on WWTP influent phosphorous concentrations of 2.0 mg/l and overflow values of 2.8 mg/l based on data generated in the late 1970s.

WWTP data for the summer/fall periods, July to September, 1978 and July to October, 1979 were separated according to dry and wet weather conditions and with and without tertiary treatment (pickling liquor or FeSO4 addition) and total phosphorous removal efficiencies were computed. Results were as follows:

%Removal of Total Phosphorous

	Secondary Treatment	Tertiary Treatment
Dry Weather (No. days)	60 (21)	69 (151)
Wet Weather (No. days)	56 (9)	69 (34)
Average	59	69

The above results indicate a 9% to 13% removal increase using tertiary treatment. Although a 4% decrease in total phosphorous removal occurred during wet weather secondary treatment, no change was noted from dry to wet conditions with tertiary treatment.

A review of WWTP data obtained for the summer/fall period June to September, 1984 indicates an average total phosphorous removal efficiency of 54% (120 days) for combined dry and wet weather conditions without secondary treatment. Beginning on September 17, 1984 both total and soluble forms of phosphorous were investigated. In addition, on October 7, 1984 teritary treatment (pickling liquor addition) was re-instated allowing for analysis of the removal efficiency for both total and soluble phosphorous while under optimum treatment conditions. Resulting data shown in Table 6.9 indicate a 50% total phosphorous removal efficiency with secondary treatment which is 9% below 78/79 data and 4% below June to September 1984 average data. No removal of soluble phosphorous was noted. After the renewal of tertiary treatment, a 70.4% removal efficiency was attained.

Inspection of the results in Table 6.9 indicates that total phosphorous removal with secondary treatment is limited to the removal of the non-soluble phosphorous fraction. On the average 95.5% of the non-soluble fraction was removed. Soluble concentrations actually increased an average of 4%. Considering this removal efficiency and that the influent phosphorous is 54% nonsoluble on average, the 50% total phosphorous removal value reported above is normally attained. Inspection of the results in Table 6.11B indicates that tertiary treatment provides removal of both the soluble and non-soluble fractions. Non-soluble removal was 88%, and the soluble removal increased to 47.4%. Influent phosphorous concentrations were 45% soluble (similar to the 45.9% reported above - see Table 6.11A) such that the total efficiency of 70.4% was attained.

In conclusion, tertiary treatment increases the soluble fraction removal by 47%, and reduces the non-soluble removal by 7.5%, resulting in a total phosphorous removal of 70.4% or 20.6% above that attained without special treatment. The decrease in non-soluble phosphorous removal (95.5% as compared to 88%) is apparently due to incomplete secondary clarifier settling of the floculant material generated by the chemical reaction. Soluble phosphorous tied up in the unsettled floc would appear as non-soluble phosphorous in the laboratory analysis. Part of the soluble phosphorous removal rate reported is attributable to this apparent increase in the non-soluble fraction and on a total phosphorous basis the effects tend to cancel each other out.

Results of these analyses were used to evaluate potential WWTP efficiencies and therefore the net possible reductions in river loadings under the various treatment schemes. Values of 50% and 75% removal were selected to represent annual averages of total phosphorous removal efficiencies for conditions with secondary and tertiary treatment, respectively.

TABLE 6.9

WWTP Phosphorous Removal Results (1984)
Effects of Treatment With/Without Pickling Liquor (FeS04)

A. Treatment Without FeSO4 (17 Days)

Influent	Avg. conc. mg/l	% of Total
Total P	2.70	100.0
Soluble P	0.95	45.9
Non Soluble P	1.12	54.1
Effluent		
Total P	1.04	100.0
Soluble P	0.99	95.2
Non Soluble P	0.05	4.8

% Removal	
Total P	49.8
Soluble P	-4.0
Non Soluble P	95.5

B. Treatment With FESO4 (8 Days)

Influent		
Total P	1.69	100.0
Soluble P	0.76	45.0
Non Soluble P	0.83	55.0
Effluent		
Total P	0.50	100.0
Soluble P	0.40	80.0
Non Soluble P	0.10	20.0

% Removal	
Total P	70.4
Soluble P	47.4
Non Soluble P	88.0

CHAPTER 7

DESIGN OF REDUCED BMP PLAN MODIFICATIONS

7.1 Foreword

Throughout the preliminary Phase 1 modeling efforts (1979-1981), simulations were run varying the hydraulic operations of regulator chambers based on general physical modification concepts. Design of the exact physical changes required within each chamber was completed during the design of the reduced BMP Plan implementation (Phase 2).

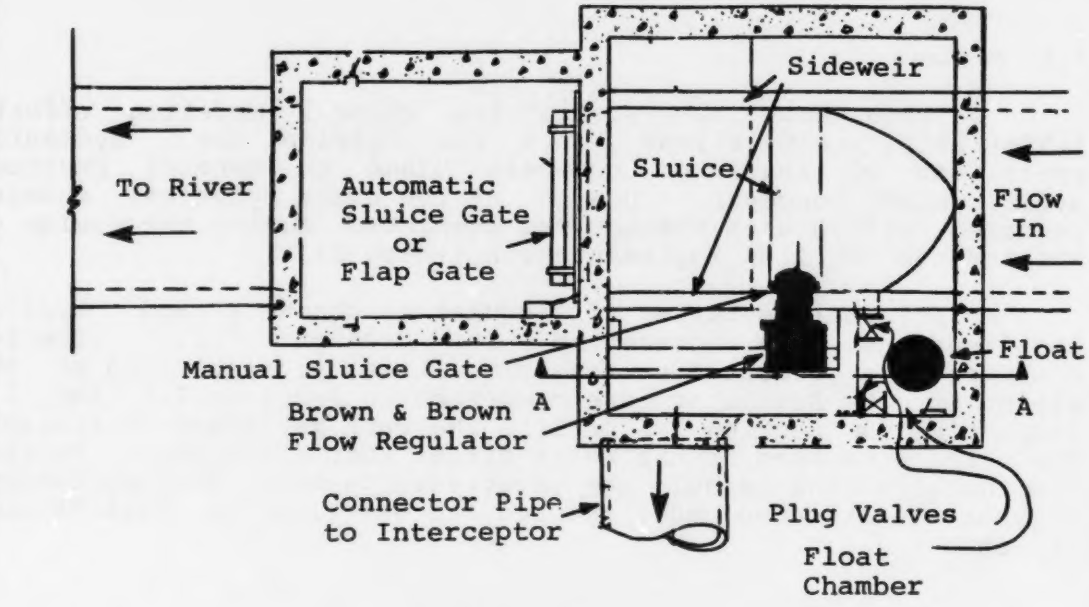
Physical details of regulator chambers and typical modifications are presented in Section 7.2. Special modifications required at the Adams Street Regulator and at the Weiss Street Regulator are presented in Sections 7.3 and 7.4 respectively. Section 7.5 details the work performed to increase the effective head at the Weiss Street Pump Station. Section 7.6 details the single new upstream in-line storage/control chamber constructed under the reduced BMP Plan at Salt/Vermont Streets.

7.2 Regulator Chamber Details and Modifications

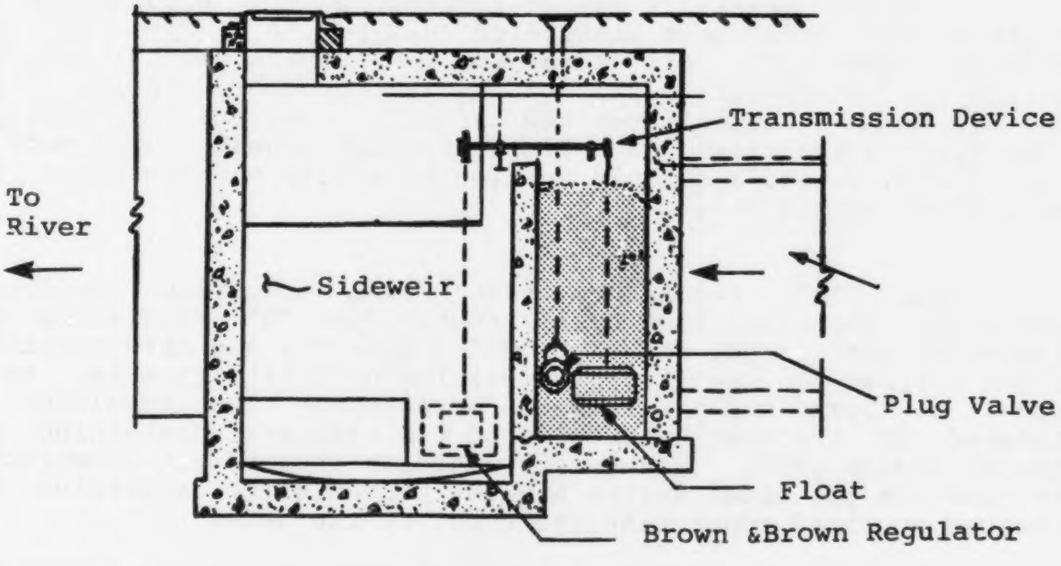
Three general types of regulators were encountered in Saginaw and have been classified as Type "A", Type "B" and modified Type "B". All regulators are underground concrete structures consisting mainly of the four concrete walls, floor and ceiling, a central open top conduit, concrete side weirs, a flap gate to the river, a wall cavity with manual sluice gate in one side weir and a conduit leading to a drop manhole above the main intercepting sewer.

Type "A" regulators have fixed underflow orifices, generally about 1/2 foot by 1/2 foot. Type "B" regulators are typically much larger than Type "A" regulators and also contain a Brown & Brown flow regulator consisting of a wall thimble, swing gate, a float and transmission apparatus. The regulator is located on the downstream side of the side weir containing the manual sluice gate. A separate adjacent chamber was constructed to contain the float device and the transmission apparatus was located overhead connecting the float to the gate.

Figure 7.1 illustrates the layout of a typical Type "B" chamber. Type "A" chambers are similar without the float chamber and Brown and Brown regulator as depicted in Figure 7.2. Figure


Sideweir
Sluice
To River
Automatic
Sluice Gate
or
Flap Gate
Flow
In
Float
Manual Sluice Gate
A
A
Brown & Brown
Flow Regulator
Connector Pipe
to Interceptor
Plug Valves
Float
Chamber

Plan View


Transmission Device
To
River
Sideweir
Plug Valve
Float
Brown &Brown Regulator

Section View A-A

Figure 7.1 Type "B" Regulator Chamber Schematics

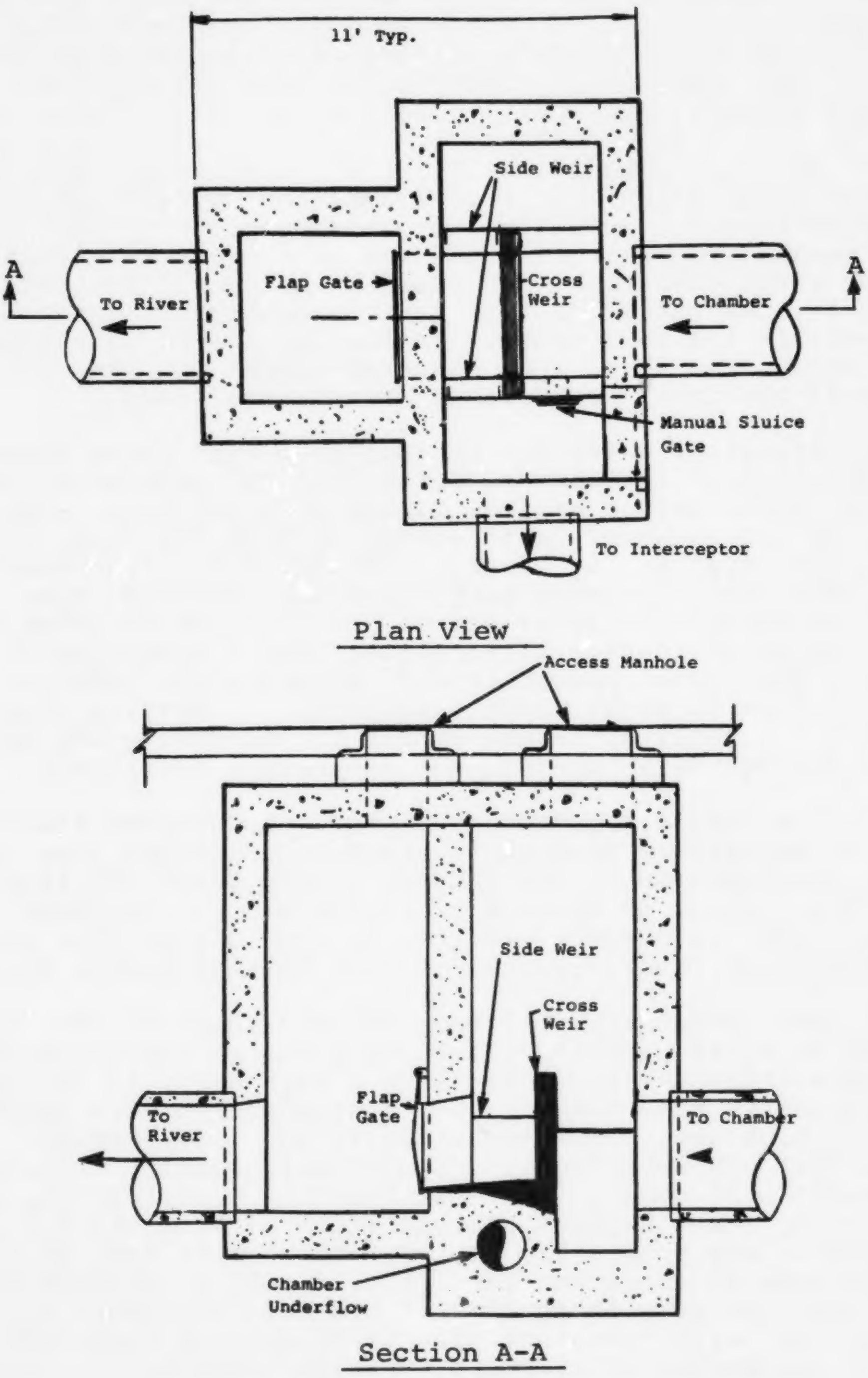

11' Typ.
Side Weir
A
A
To River
Flap Gate
Cross Weir
To Chamber
Manual Sluice Gate
To Interceptor
Plan View
Access Manhole
Side Weir
Cross Weir
Flap Gate
To River
To Chamber
Chamber Underflow
Section A-A

Figure 7.2 Type "A" Regulator Chamber

7.3 is a plan view of a Type "B" chamber depicting flow patterns. Type "B" regulators have variable underflow rates altered by a float gate operation. As water levels rise, underflow orifices close. Table 7.1 includes pertinent regulator chamber dimensions used in the hydraulic calculations and Table 7.2 includes pertinent original and modified weir data.

In the unmodified Type "B" and Type "A" regulator chambers high flows not handled by the underflows would be discharged through backwater gates to the river. The operation of the backwater gates was dependent on their seat elevations and relative river elevation, and could be prevented or hindered from opening in some cases for high river conditions. Most backwater gates were in the same chamber as the regulator. A few backwater gates were separated and situated closer to the river (for example at the Cronk and Genesee Street Regulators).

Backwater gates are typical flap-type swing gates (with the exception of the five modified Type "B" regulator chambers). In most cases only a minimal amount of storm flow increase is needed to raise water depth enough to open the flap gates and cause a discharge to the river. The gates are also sensitive to river elevations as most gate seats are located near average river elevations. The Weiss Street gravity line is notably about five feet below average river elevations. High river flows can restrict the gate operation and subsequently require higher depths of flow to cause river discharges. Varying river stage can have a somewhat random effect in increasing the amount of in-line storage of storm flow and decreasing overflows.

The third group of regulators encountered are the five Type "B" regulators located in the Hancock Street area modified during construction of the Hancock Street Detention Facility in 1977. The intent of the modifications was to increase in-line storage, and to divert flow in excess of the in-line storage to the interceptor (and eventually to the Hancock Street Facility).

The regulators had been altered to cause the underflow orifices to close rapidly, limiting flows to the interceptor and thus committing the first flush of a rain event to in-line storage created at the chamber. EDP considered it more advisable to maintain high underflow discharges to the interceptor, limited only to what the interceptor can maximally handle. Operation in this manner will commit the first flush of a rain event, generally carrying the highest concentration of pollutants, to the inteceptor, and eventually to the WWTP. This mode of operation is preferable to capturing the "first flush" in storage which may become overflow at a later time if the rain continues and can be accomplished with constant discharge devices that are more reliable and easier to maintain than the present float mechanisms.

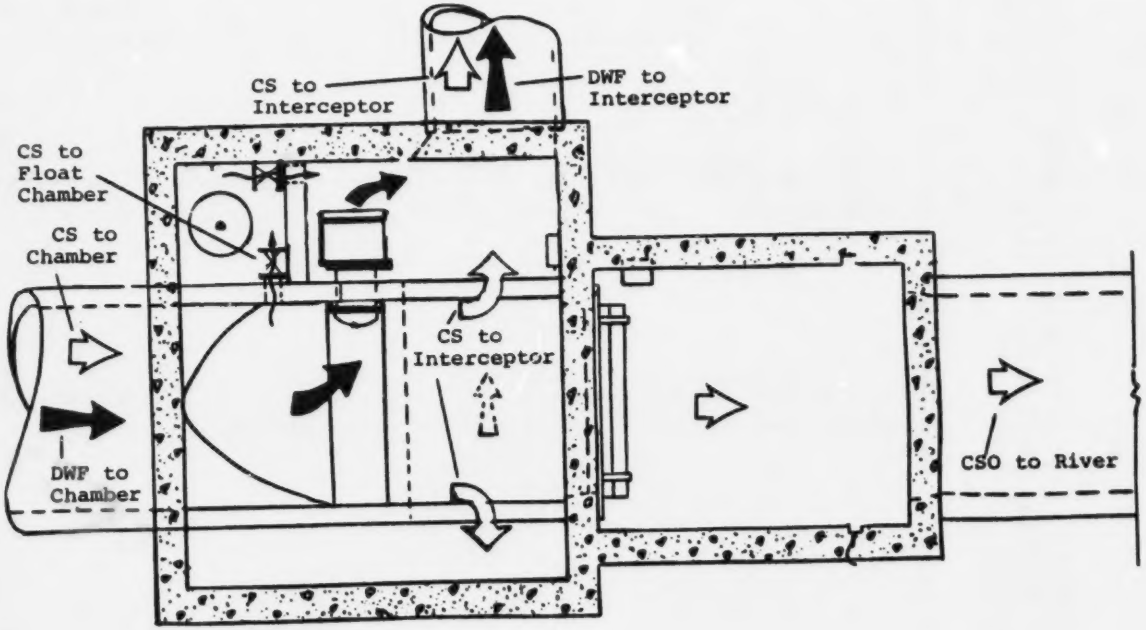

CS to Interceptor
DWF to Interceptor
CS to Float Chamber
CS to Chamber
DWF to Chamber
CS to Interceptor
CSO to River

Figure 7.3 Type "B" Regulator Chamber Flow Patterns

TABLE 7.1

BMP Program Regulator Chamber Weir Alterations

LOCATION	ELEMENT OF REDUCED PROGRAM	EXISTING WEIR LEVEL ABOVE INVERT FT.	BMP WEIR LEVEL FT.	EXISTING OVERFLOW WEIR FT.	NEW OVERFLOW WEIR FT.
REMINGTON	Y	5.0	2.16	-	6.0
THROOP	Y	4.4	3.17	-	5.4
FITZHUGH		6.9	5.17	-	6.42
JOHNSON		3.96	.96	-	2.7
FEDERAL		4.36	1.2	-	2.9
W. GENESEE	Y	5.78	.18	-	1.6
MACKINAW	Y	3.4	.94	-	4.4
HAYES	Y	2.94	.14	-	1.2
CARLISLE		3.84	.37	-	1.4
E. GENESEE		3.76	.5	-	1.5
MILLARD		2.58	1.09	-	2.65
HOYT		3.15	1.3	-	2.95
HOLDEN		3.67	0	-	1.0
HOLLAND		4.35	1.09	-	2.7
EMERSON		3.9	1.34	-	3.4
WEBBER		0	8	-	3.25
McCOSKRY		8.2	4.16	-	6.5
ADAMS	Y	5.67	2.94	-	5.28
JANES		5.2	.5	-	2.0
THOMPSON		7.4	6.5	-	8.5
CRONK	Y	6.65	3.28	-	4.95
FRASER	Y	9.0	9.0	-	5.67
WEISS	Y	0	10.0	-	9.0
FOURTEENTH		0	4.8	-	2.8
FIRST		3.42	.5	-	2.5
MILLER		2	2	2	3.6
AMES	Y	1.25	1.25	1.25	3.75
COURT	Y	1.1	1.1	2.8	2.8
CASS		1.3	1.3	2.0	2.0
VANBUREN		.9	.9	1.9	1.9
DEARBORN	Y	.9	.9	1.9	1.9

TABLE 7.2

REDUCED BMP PROGRAM REGULATOR CHAMBER AND FLOW REGULATION DATA

Name/Location	Type	Chamber Dimensions Length**	Chamber Dimensions Width Between Side Weirs***	Flow Regulator Design Criteria Discharge
		Ft.	Ft.	CFS
Remington	B*	12'6"	6'0"	1.56
Throop	B*	12'6"	6'0"	1.56
W Genesee	B	18'6"	4'0"	2.9
Mackinaw	B*	12'6"	5'3"	1.74
Hayes	B*	9'6"	4'0"	2.0
Adams	B	30'9"	7'4"	10.38
Cronk	B	12'6"	6'0"	8.8
Fraser	B	-	-	24.0
Weiss	B	5'6"	2'2"	22.2
Ames	A	4'	3'6"	.37
Court	A	4'	2'9"	.3
Dearborn	A	4'	2'4½"	.3
Salt/Vermont	Upstream Chamber			

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* Modified Hancock Area Regulator
** Length of existing weirs to be altered (see Table 7.1)
*** Length of new weirs to be constructed (see Table 7.1)

Selection of appropriate vortex valves for each regulator chamber location modified under the reduced BMP program was based on apportioned flow rates (six times average dry weather flow - Table 7.2) considering WWTP capacity and modifed chamber configurations. Vortex valve head-discharge curves characteristically include low level orifice peak, switchback and near constant (low rate of flow increase) vortex flow condition. Vortex valve selection was accomplished by comparing a particular chamber's design flow/perceived operating head range with manufacturer's rating stage/discharge data. The most desireable selection results when the chamber's operating head range extends from at least low head orifice peak for the device up to conditions where full vortex braking occurs. A less desireable but still acceptable choice (if minor flucuations can be tolerated) occurs when the upper end of the configuration's head range does not fall within the devices establihsed vortex operating range but in the switchback zone. The design stage/discharge characteristics for each of the vortex valves is listed in Table 7.3.

The regulators were modified in the reduced BMP program to ensure that flows in excess of what is discharged through the underflow regulating devices or stored in the system behind the weir, will first overflow the side weir and be directed to the interceptor. Only extremely high flows will overflow the new cross weir separating the backwater gate portion of the chamber and be directed to the river. This weir arrangement was considered by EDP to be both the least expensive and most available method of limiting oveflows at any of the regulator chambers in the system.

7.3 Proposed Weir Alterations

The reduced BMP program included the alteration of 12 existing regulator chambers. Each of these regulator chambers is located on the West Side of the Saginaw River. Table 7.1 lists all the existing regulator chambers and the proposed weir alterations under the complete BMP program. Regulator chambers altered under the reduced BMP plan are noted. Typical details of regulator chamber alterations were shown in Chapter 1, Figure 1.6.

Regulator modifications included construction of a concrete cross weir wall between the backwater gate and the flow channel sluiceway of sufficient elevation to insure that stormwater volume in excess of available in-line storage capacity flows to the interceptor with only extreme flows discharging to the river. The new cross weir is located just downstream of the

TABLE 7.3
Vortex Valve Stage/Discharge Characteristics*
Discharge (l/s)

Head (ft)	A	B	C	D	E	F	G	H	I
1				68	50	38	40	10	4+
1.5		240	190	77	57	44	48	7	5
2	470	265	230	82	51	39	42	8	5+
2.5	530	285	250	72	44	31	33	9	6+
3	570	300	230	79	46	29	34	10	7
3.5	600	285	190	87	48	31	35	11	7+
4	620	260	180	95	51	33	37	12	8+
4.5	600	255	175	102	54	35	40	12+	8+
5	570	270	180	108	58	36	44	13	9+
5.5	550	285	195	113	62	38	47	13+	9+
6	600	300	200	119	64	40	50	14	10+
7.	658	325	220			43	55	15	
8.0	710		240						
9.0	760		260						
10	804								
11	850								
12	891								

(1 cfs = 28.3 l/s)

A - Weiss
B - Adams
C - Salt/Vermont
D - W. Genesee
E - Hayes
F - MacKinaw
G - Remington/Throop/Cronk
H - Ames
I - Court

* As per data provided by Advanced Fluidics, Inc.

dry weather sluiceway and extends across the chamber between the existing side weirs. Lengths of the new cross weirs are indicated in Table 7.2. Existing side weirs creating the in-line storage were modified to ensure that the water surface elevation due to backwatering and flow dynamics satisfied the arbitrary criteria of a 10 foot buffer zone to the ground elevation in the area. The side weirs generally extend the length of the regulator chamber and are modified the entire distance. Side weir lengths are included in Table 7.2. Extreme stormwater flows in excess of the interceptor capacity will backwater the regulator chamber and spill over the side weirs downstream of the new cross weir and discharge to the river.

An inter-active computer program was developed to determine optimum crest elevations of the modified side weirs and new cross weir. Typical storm flows (2 and 5 year design storm), extreme flows (50 year storm), weir lengths, and chamber outlet capacity and other physical dimensions were considered in the program. Conceptually it was intended to select a modified configuration wherein maximum storage would be provided prior to side weir spillover to the interceptor. In addition, maximum flow to the interceptor would occur prior to bypass to the river. The cross weir elevation was selected such that flow would top this weir at the same time bypass to the river occurred due to interceptor capacity limitations. The need to ensure sufficient unrestricted passage for extreme storm flows was the limiting factor in maximizing storage potential. Due to the uncertain nature of the actual flow hydraulics under probable turbulent conditions within the chamber, conservative weir elevations were selected.

Four locations were designated to include adjustable side weirs and cross weirs accomplished by attaching moveable stainless steel plates to the concrete weirs. Based on operating data within the chamber and stormwater backup conditions upstream of the chamber, these plates can be adjusted to increase storage if excess surcharge does not appear probable.

During the 1977 Hancock Street Facility Implementation the flap gates were removed at the West Remington Street and at the Throop Street Type "B" regulators, and were replaced with automatic sluice gates. These flap gates were re-installed under the reduced BMP plan. Both flap gates and automatic sluice gates exist at Adams Street, Mackinaw Street and Hayes Street. Under the "reduced BMP" plan implementation, all automatic sluice gates were raised and left in the full open position at these locations.

In general the scope of work at Type "A" chambers at Ames Street and Court Street included: a) removal of existing regulator slide gate, gate stem, supports, brackets anchor bolts and appurtenances; and, b) installation of new vortex valve regulating device and accessories.

Modifications to the Type "A" Regulator at Dearborn Street included: a) adjusting the existing slide gate to a 6 inch vertical opening, b) cutting down walls and resurfacing crest on existing side weirs; and c) constructing new cross weir.

The general scope of work at type "B" chambers included: a) removing existing float operated mechanical regulators including float, gate, operating mechanism, cables, supports, anchor bolts and appurtenances; b) removing existing stainless steel weir plate and supports, c) installing new vortex valve flow regulating device and accessories, d) cutting down walls and resurface crest of existing side weirs, e) constructing new cross weir, f) installing adjustable stainless steel weirs on the two modified side weirs and on the new cross weir; and g) recontouring floor sections with concrete fillets to prevent debris buildup and direct flow.

In addition to the above items at Type "B" regulator chambers at West Remington Street and at Throop Street, other modifications included: removing and salvaging the existing automatic sluice gates and installing new tide gates.

7.4 Special Modification of the Adams Street Type "B" Regulator

The placement of the new flow regulating devices within most Type "B" chambers was behind the side weir wall next to the interceptor outlet (See Chapter 8 for additional details). The installation of the new flow regulator could not be similarly accomplished at the Adams Street Regulator due to the relative flow regulator size and the chamber size. Considering the very large drainage area serviced by the Adams Street Chamber the new flow regulator was roughly twice the size of other "B" type chamber devices.

The chamber area downstream of the side wall containing the original Brown and Brown regulator is 4 ft. wide and due to this limited space, it was necessary to locate the new flow regulator directly in the sluice area of the chamber. As with all other chambers, the design of the final configuration was to include the following functions: a) allow dry weather flow to pass through the flow regulator to the interceptor; b) store initial wet weather flow; c) pass additional wet weather flow to the interceptor; and, d) to limit river bypasses to extreme flow conditions.

Only one of the typical two side weirs existed at the Adams Street Chamber which limited the weir length available for stormwater overflow to the interceptor prior to overflow to the river. The need for additional side weir length coupled with the difficulty in placing the new flow regulator and a desire to minimize debris accumulation in remote chamber areas led to a complete modification of the chamber interior.

Figure 7.4 illustrates the final configuration selected. Two new cross weirs were constructed in the chamber. The upstream cross weir was placed at an angle to the side walls for the dual purpose of maximizing the available overflow length and to direct flow to the regulator device inlet. This weir was constructed at the same elevation as the modified, (cut down) side weir such that storm flow in excess of storage capacity will top these weirs simultaneously and pass to the interceptor.

Concrete fillets were used upstream of the cross wall to assist directing flow to the inlet and prevent debris accumulation. Downstream of the cross wall, the sluiceway was concrete filled and the existing regulator device and the manual side wall sluice gate were removed. No overflow weir existed at this side wall and to allow passage of new regulator device outflow and cross wall overflow, a section of the side wall was removed. A wall cut of 5 ft by 7 ft was selected based on both flow volume criteria and to accommodate workman passage between chamber sections.

Concrete filling of the original sluiceway flush with the chamber inlet floor provided the large area necessary for installation of the new flow regulator. A cast-in-place wall thimble was designed for inclusion in the upstream cross weir for mechanical attachment of the flow regulator. In addition, the device was secured to a cast-in-place floor bracket. Again, concrete fillets were designated to minimize areas of debris deposition and direct flow to the outlet.

Control of flow to the river is accomplished by the downstream cross weir and the original side weir section downstream of this cross weir. The intent is that all flow up to the interceptor capacity be stored or pass either through the 5 ft by 7 ft opening or over the side weir to the interceptor. Storm flows greater than the interceptor capacity would top both the downstream cross weir and the downstream section of the side weir, pass through the backwater flap gates and to the river. Modification of the downstream side weir section and construction of the downstream cross weir were designed at the same elevations as the upstream cross weir and modified upstream side weir. This

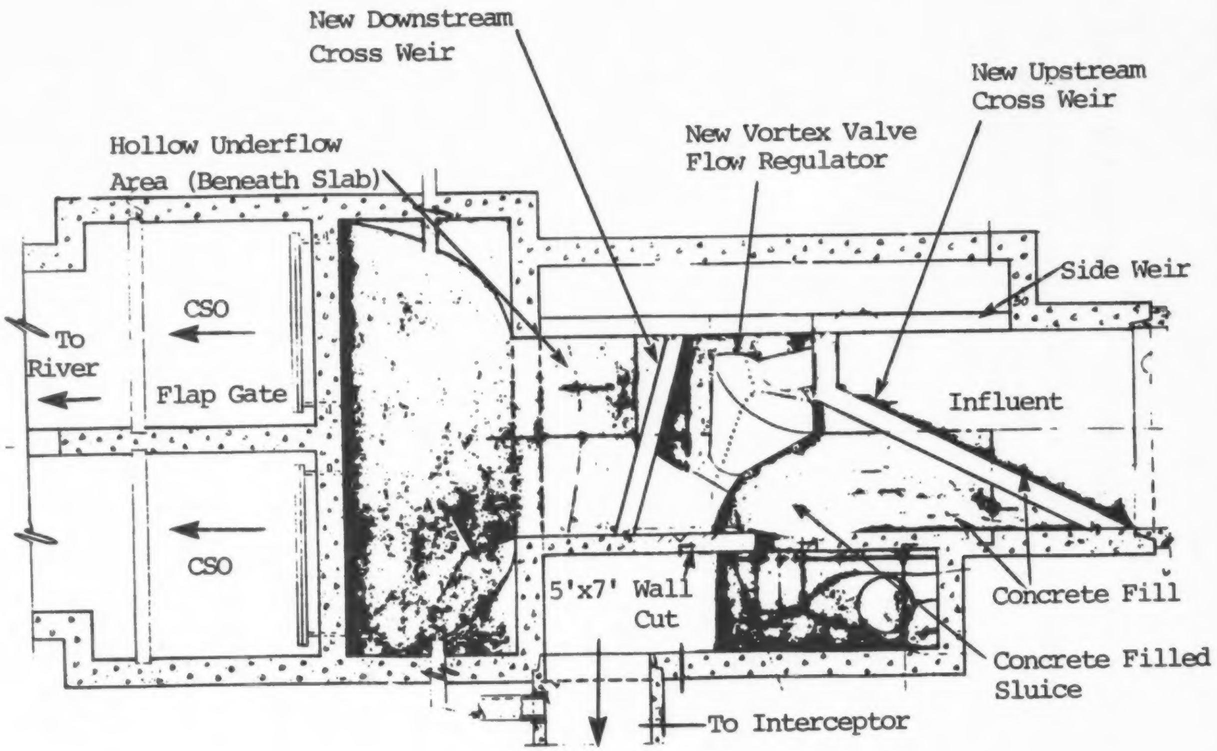

New Downstream
Cross Weir
New Upstream
Cross Weir
New Vortex Valve
Flow Regulator
Hollow Underflow
Area (Beneath Slab)
Side Weir
CSO
To
River
Flap Gate
Influent
CSO
5'x7' Wall
Cut
Concrete Fill
Concrete Filled
Sluice
To Interceptor

Figure 7.4 Adams Street Regulator Chamber Modification

elevation, 2.4 feet below the original side weir elevation, and 1 foot above the backwater gate seat elevation was calculated to be optimum to maximize storage and provide sufficient clear passage to the river under extreme flow condiitions.

7.5 Special Modification of the Weiss Street Regulator Chamber

Flow originating upstream of the Weiss Street area is regulated within a small structure located between the 120 inch gravity line serving the area and the Weiss Street Pump Station. Unlike most other "B" type regulators, this location contains no side spillover weirs (See Figure 7.5). Flow rates above the regulated allowance will backwater the chamber, the inlet to the chamber pipe and the 120 inch gravity line. Flap gates are located at the end of the 120 inch line with seat elevations 3.6 feet above the invert. Stormwater storage above this level can pass to the river depending on the river elevation. The average river elevation is 2.09 feet above the gate seat such that a normal storage depth of about six feet within the 120 inch line is attained prior to bypass to the river.

Downstream of the regulator chamber a 36 inch pipe leads to the diversion chamber section of the Weiss Street Pump Station. Details of that location are included within the Weiss Street Pump Station modifications discussed in the next section.

Modifications to the Weiss Street Regulator were limited to removal of the existing float/gate type regulating device, and replacement with a vortex type unit and minor structural changes. These modifications hydraulically altered the flow through pattern from one of decreasing flow to the interceptor with increased depth of backwatering to a situation wherein flow increased with backwatering to a rate of six times the dry weather rate. This maximum rate is based on the WWTP capacity to normal flow ratio and was used at all modified regulators.

As with the Adams Street Regulator the area served by the Weiss Street Regulator is quite large and the size of the regulating device is about twice the size of those at other locations. Both the 120 inch gravity line and the pump station diversion chamber were deemed unsuitable for flow regulation such that installation options included modification of the regulating chamber interior and exterior walls or placing the unit in the existing space.

Although the space within the existing regulation chamber was extremely limited and required placing the outlet orifice section in the outlet pipe and removal of one interior wall section, this option was considered the most cost effective.

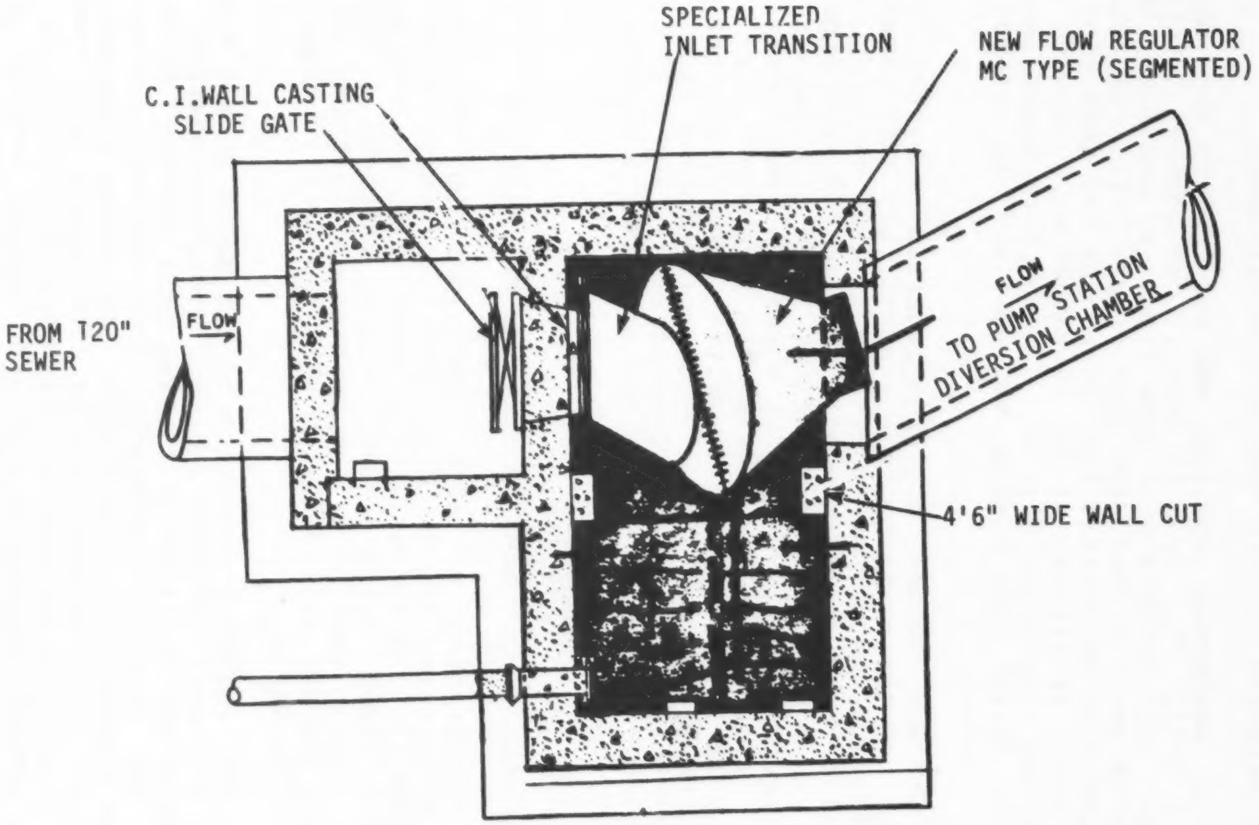

SPECIALIZED
INLET TRANSITION
NEW FLOW REGULATOR
MC TYPE (SEGMENTED)
C.I.WALL CASTING
SLIDE GATE
FROM 120"
SEWER
FLOW
FLOW
TO PUMP STATION
DIVERSION CHAMBER
4'6" WIDE WALL CUT

Figure 7.5 Weiss Street Regulator Chamber - Plan View

A specialized adapter was required for the dual purpose of transforming the flow pattern from the existing rectangular regulator wall thimble to the required round inlet configuration and to offset the unit for proper alignment of the outlet orifice with the chamber outlet pipe. Unit performance would not be adversely affected by this mode of installation.

An emergency spillway was provided in the chamber by cutting a 3-1/2 ft. by 5 ft hole in the chamber interior wall above the wall thimble at a base elevation equal to the crown elevation of the 120 inch pipe (See Figure 7.6). If extreme high river elevation conditions surcharged the 120 inch pipe during a storm event, spillover at the regulator wall would commence, bypassing the regulating device. Excess flow would then pass to the Weiss Street Pump Station wet well and be pumped to the river.

Storage volume at this location remained unchanged. Alteration of the existing river outlet flap gate assembly was not a practical method of increasing in-line storage considering the relative normal river elevation to the 120 inch pipe location and the need to ensure that extreme storm flow passage to the river. Benefits of the BMP program implementation at this location included: a) replacing the float/gate type regulator with a near maintenance free vortex type regulator, and b) altering the storm period hydraulic configuration from one which limits the passage of "first flush" flows to the WWTP to a system which passes "first flush" flow and maintains a rate as allocated by the WWTP total capacity.

7.6 Weiss Street Pump Station Wet Well Modification

Simulations of flow conditions in the interceptor system demonstrated the value of altering conditions at the Weiss Street Pump Station wet well to produce greater cross river flow rates and reduce Weiss Street Station pumpage. Optimization analysis of increasing the water surface elevation required to spill into the wet well dictated that a 5 foot increase would be the most effective.

The existing structure at the Weiss Street Pump Station included a circular cassion wet well diversion chamber. Most of the cassion was utilized as the wet well serving three 100-cfs pumps. One section, separated from the wet well acted as a diversion chamber. All West Side Interceptor flow and regulated Weiss Street 120 inch gravity line flow enters this chamber. Dry weather flow exits the chamber through a 36 inch sluice gate to a 48 inch conduit leading to a 42 inch conduit beneath the Saginaw River connecting to the East Side Interceptor. A 12 inch

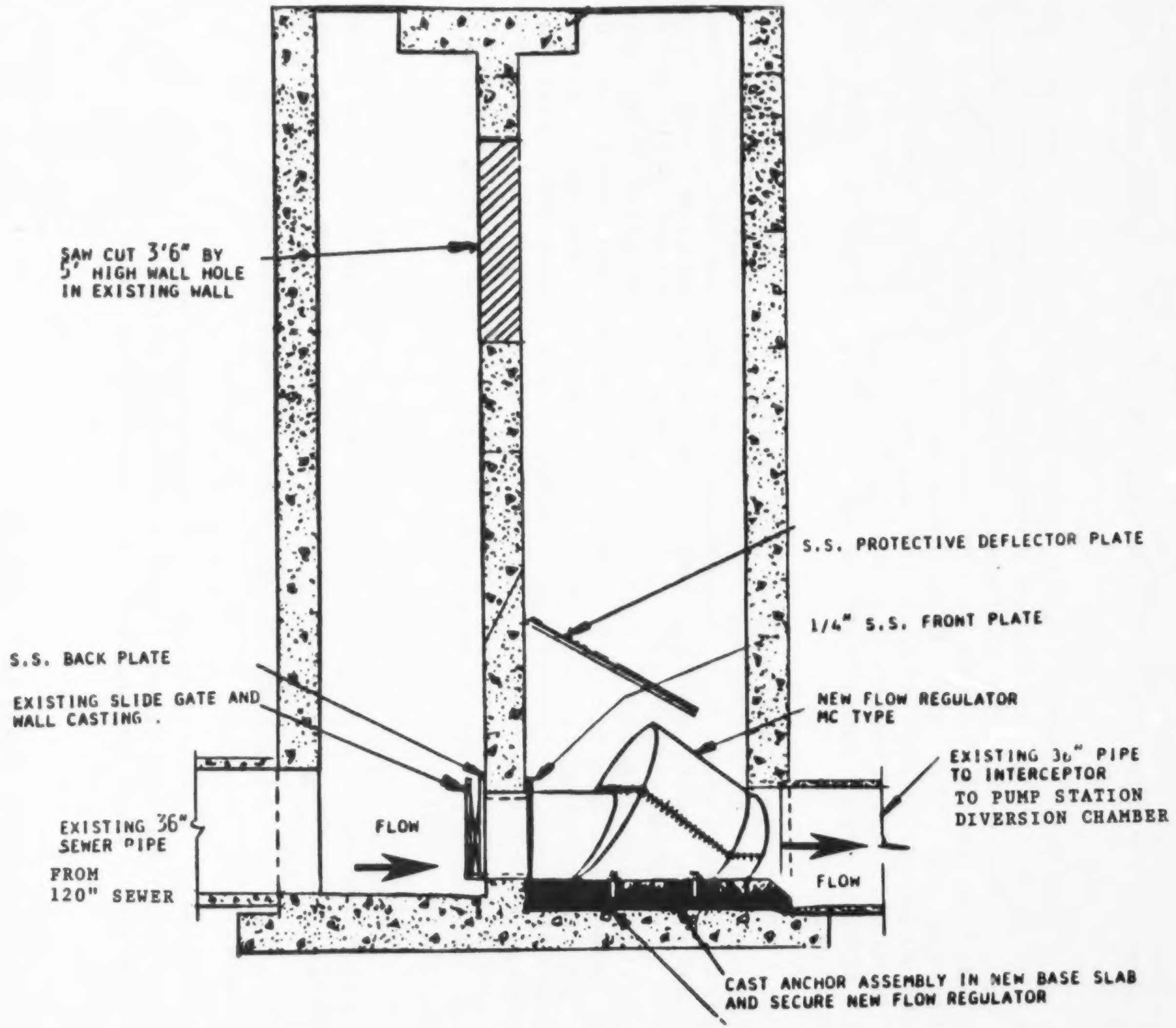

SAW CUT 3'6" BY
5' HIGH WALL HOLE
IN EXISTING WALL
S.S. PROTECTIVE DEFLECTOR PLATE
1/4" S.S. FRONT PLATE
S.S. BACK PLATE
EXISTING SLIDE GATE AND
WALL CASTING .
NEW FLOW REGULATOR
MC TYPE
EXISTING 36" PIPE
TO INTERCEPTOR
TO PUMP STATION
DIVERSION CHAMBER
EXISTING 36"
SEWER PIPE
FROM
120" SEWER
FLOW
FLOW
CAST ANCHOR ASSEMBLY IN NEW BASE SLAB
AND SECURE NEW FLOW REGULATOR

Figure 7.6 Weiss Street Regulator Chamber Modifications-Section View

concrete wall separates the diversion chamber and wet well areas and contains two 4-foot by 5-foot flap gates with seat elevations 2.75 feet above the diversion chamber floor (See Figures 7.7 and 7.8).

Several possibilities were considered to increase the depth of flow in the diversion chamber prior to spillover to the wet well. These options included, in general terms, construction of a wall or baffle system in the diversion chamber, removal and relocation of the two flap gates to a higher elevation or construction of a wall within the wet well. Although space within the wet well was limited due to the location of bar screens the last option was considered to be the most feasible and cost effective.

Criteria used in the wall design and location were the following: a) increase the diversion chamber depth five feet at spillover; b) induce free discharge over the new weir wall; and, c) maintain water surface levels below the station mid floor level during maximum pump rates. The selected wall design maintained an elevation 4.75 feet above the existing flap gate seat elevation and provided sufficient length such that at the 300 cfs pump capacity flow rate (all pumps on) the water surface behind the weir wall would be one-half foot below the station´s mid-level floor elevation. Further protection was to be provided for the station´s mid-level floor by the construction of a 3-foot splash wall around the open wet well area, with splash plates at an existing stairway.

Construction of the overflow weir wall created a separate chamber within the wet well. End-of-storm event drainage of this chamber required placing a valve in the wall base. A manually operated valve was deemed the simplest and preferable to an automatic valve or a constant-open low rate orifice.

The final design allowed construction work to be accomplished during dry weather conditions with no need to divert dry weather flow or protect the interceptor from loose debris washout.

7.7 New Upstream Regulator Chamber

As part of the demonstration aspect of this project, one of the fourteen in-line storage control structures upstream of the regulators under the complete BMP program was selected for completion under the reduced BMP plan.

An existing junction chamber is located upstream of the

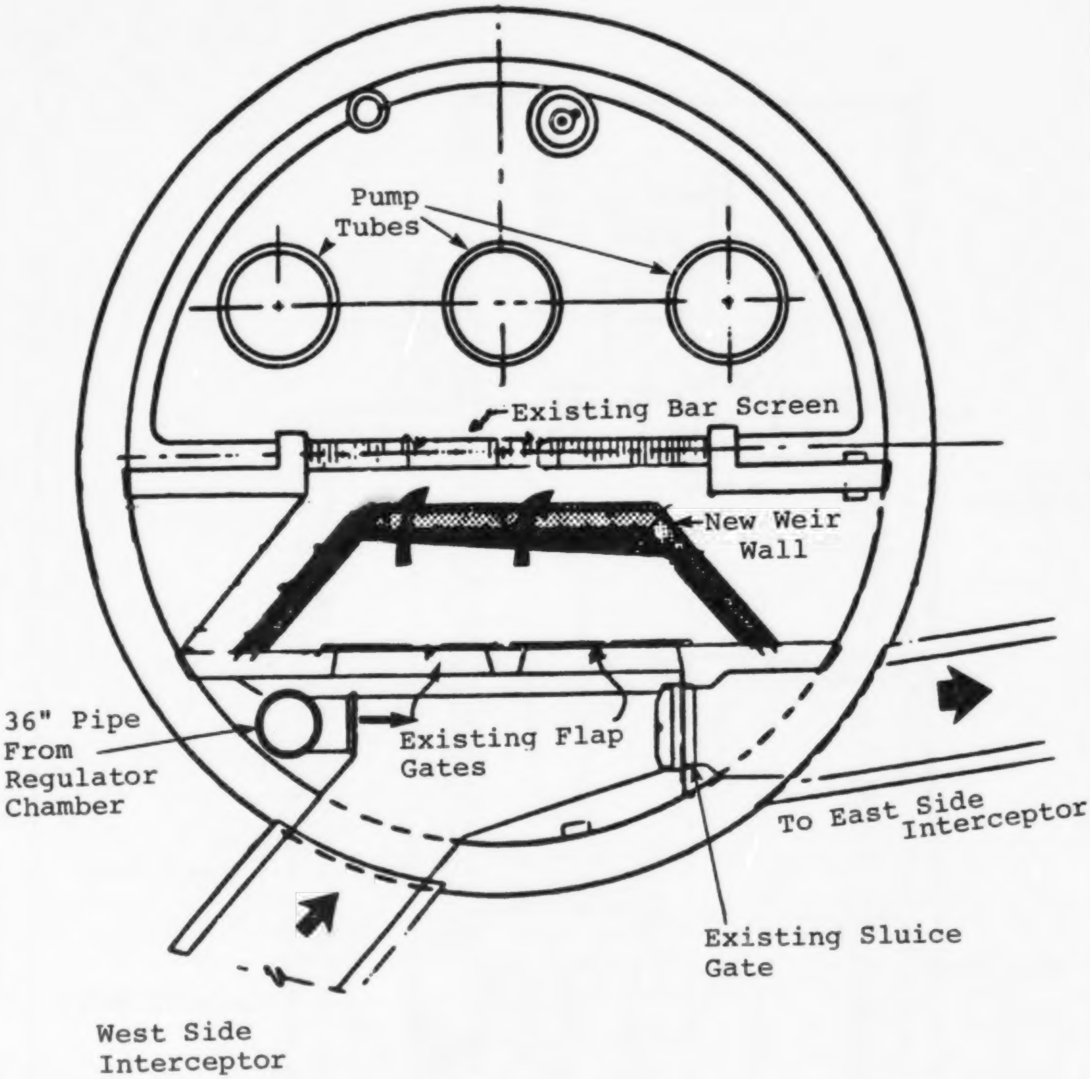

Pump Tubes
Existing Bar Screen
New Weir Wall
36" Pipe From Regulator Chamber
Existing Flap Gates
To East Side Interceptor
Existing Sluice Gate
West Side Interceptor

Figure 7.7 Weiss Street Pump Station Wet Well Modification - Plan View

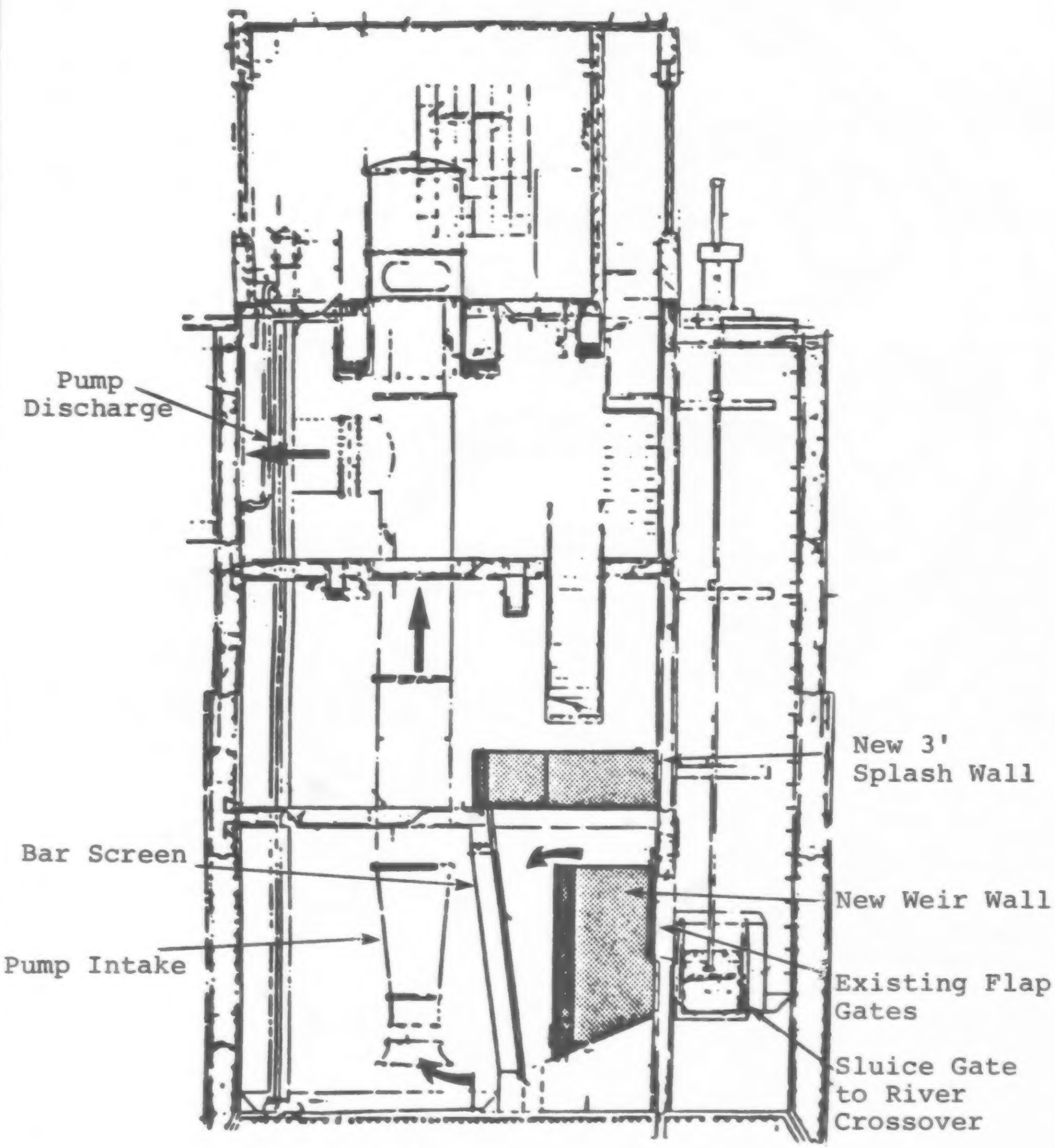

Pump Discharge
New 3' Splash Wall
Bar Screen
New Weir Wall
Pump Intake
Existing Flap Gates
Sluice Gate to River Crossover

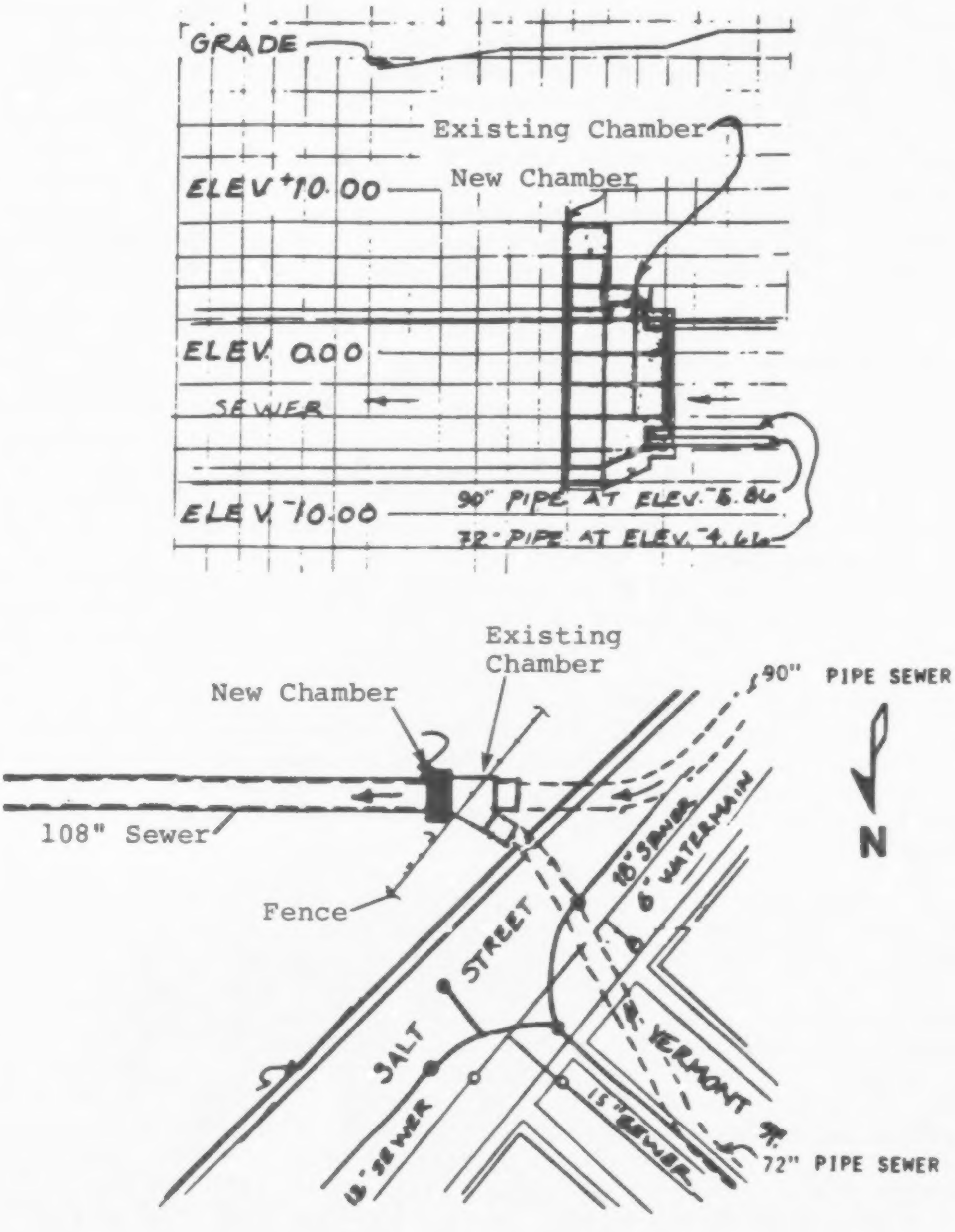
Figure 7.8 Weiss Street Pump Station Wet Well Modification Section View

regulator at Fraser Street on the West Side at the intersection of Salt Street and Vermont Street was initially selected. The chamber consists of a 90 inch reinforced concrete pipe from Salt Street joining a 72 inch reinforced concrete pipe from Vermont Street to form a 108 inch pipe. All pipes are circular with the 90 inch and 72 inch pipe inverts elevated above the 108 inch discharge pipe invert. Concrete fillets within the chamber serve to provide smooth transitions between pipes. Upon inspection, the chamber interior was deemed unsuitable for implementation of the intended stormwater storage and control devices envisioned.

A new regulator chamber was designed just downstream of this junction chamber which allows for control of wet weather flow. Locational plan and profile views are shown in Figure 7.9. Concrete weirs were constructed within the new chamber to backwater stormwater. A new flow regulating device controls the outlet rate to 6 times the dry weather flow rate. Combined sewage flow in excess of the storage capacity overflows the weir and continues to the Fraser Street Regulator. A special large diameter vent attached to the center line of the device's back plate (as can be seen in Figure 7.10) is provided for this configuration which extends up to the crest of the three-walled internal weir. When backed up sewage reaches this level within the chamber, this vent will flood causing the vortex action to cease, thereby increasing rate of flow discharging from the device. As the water level recedes below the weir crest, air will again enter the device through the vent and vortex braking will again commence. Thus this vent acts to increase the discharge from the device whenever extreme high water levels are reached.

Figures 7.10 and 7.11 illustrates the conceptual layout of the new regulator. The chamber is approximately 20 feet wide, 17 feet long and 14 feet floor to ceiling with a three wall internal weir providing 23 linear feet of overflow length. A new vortex valve flow control device is located within the bottom of the weir wall. This device arrangement allows dry weather flow to pass through the structure without undue depositon and up to six times dry weather flow to pass to the Fraser Street Regulator while backwatering most of the flow for in-line storage (175,000 cubic feet). Wet weather flow beyond the storage capacity will overtop the weir. It is estimated that the one year storm flow would crest 1.4 feet above the weir top and the 25 year storm flow would crest 2.9 feet above the weir top. The weir top is set at the crown elevation of the 108 inch pipe.

Concrete fillets were designated upstream of the weir wall to limit debris depositon and to direct flow to the flow regulator inlet. Downstream, beyond and beside the weir wall, concrete fillets were again designated to direct flow to the


GRADE
Existing Chamber
ELEV +10.00
New Chamber
ELEV 0.00
SEWER
ELEV -10.00
90" PIPE AT ELEV -5.86
72" PIPE AT ELEV -4.66
Existing Chamber
New Chamber
90" PIPE SEWER
N
108" Sewer
Fence
18" SEWER
6" WATERMAIN
SALT STREET
VERMONT
15" SEWER
12" SEWER
72" PIPE SEWER

Location

Figure 7.9 New Regulator Chamber at Salt/Vermont Streets Location and Profile

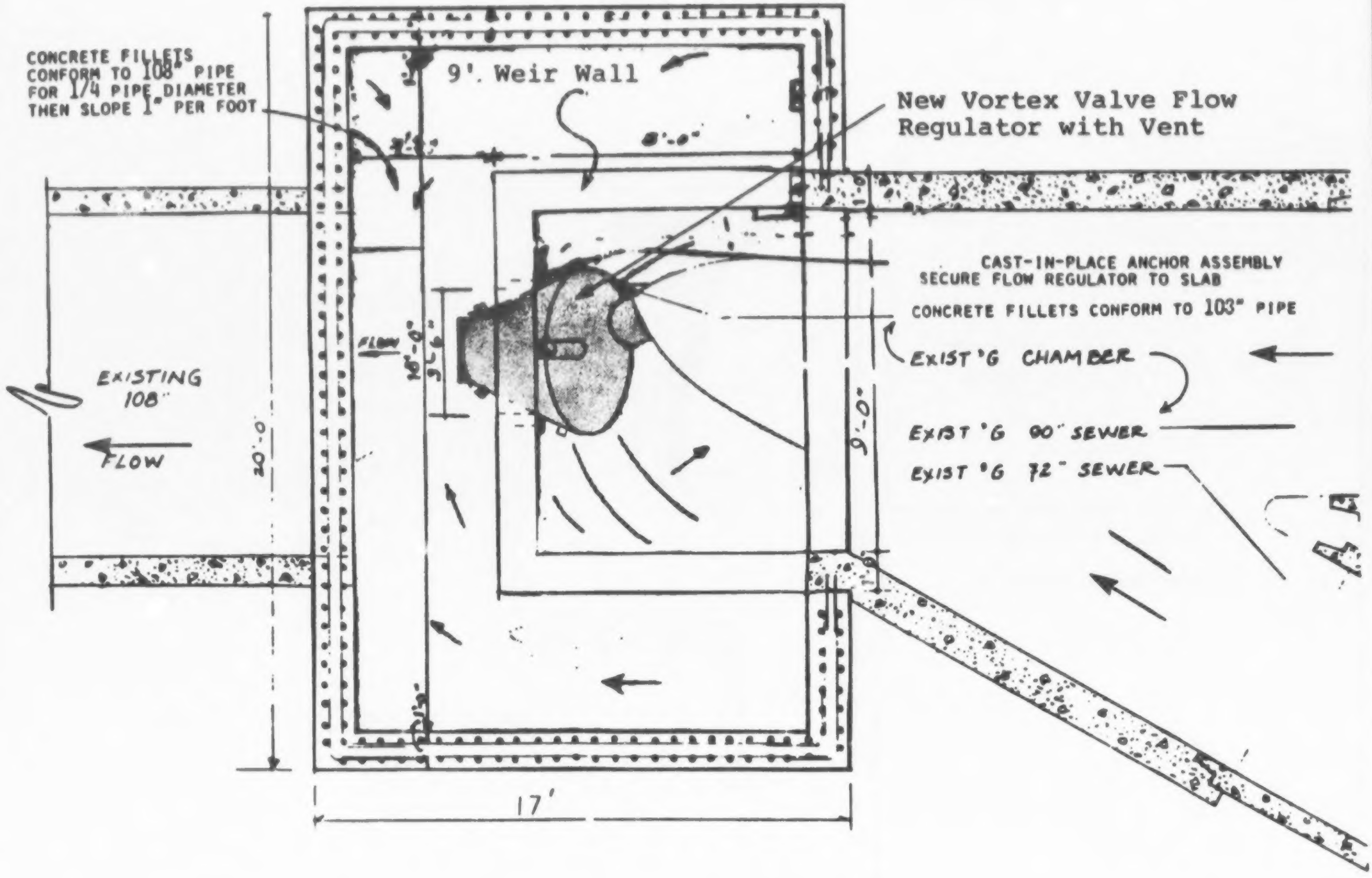

CONCRETE FILLETS CONFORM TO 108" PIPE FOR 1/4 PIPE DIAMETER THEN SLOPE 1" PER FOOT
9'. Weir Wall
8'-0"
New Vortex Valve Flow Regulator with Vent
CAST-IN-PLACE ANCHOR ASSEMBLY SECURE FLOW REGULATOR TO SLAB
CONCRETE FILLETS CONFORM TO 103" PIPE
EXIST'G CHAMBER
EXIST'G 90" SEWER
EXIST'G 72" SEWER
9'-0"
10'-0"
3'-6"
FLOW
EXISTING 108"
FLOW
20'-0"
17'

Figure 7.10 Salt and Vermont Streets Chamber - Plan View

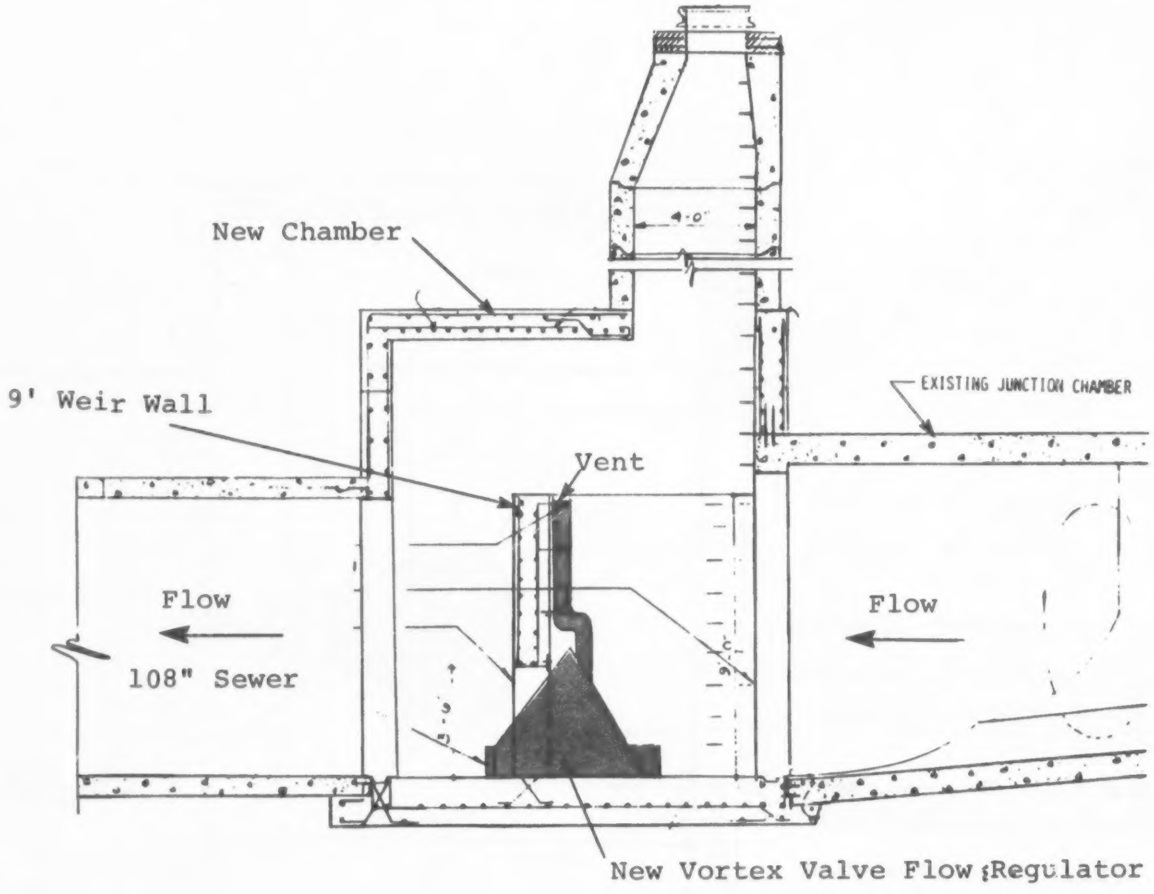

New Chamber
4'-0"
EXISTING JUNCTION CHAMBER
9' Weir Wall
Vent
Flow
108" Sewer
Flow
New Vortex Valve Flow Regulator

Figure 7.11 Salt and Vermont Streets Chamber - Section View

outlet pipe and limit deposition The ground elevation is nearly 25 feet above the level of the chamber floor. The minimum 10 foot buffer between maximum water elevation and ground surface requirement was easily attained.

CHAPTER 8

REDUCED BMP PLAN IMPLEMENTATION

8.1 Foreword

During the period of late 1983 plans and specifications were prepared for the implementation of the reduced BMP plan. The contract was awarded in March, 1984 to Gary D. Steadman Inc., Bay City Michigan . A startup meeting held on April 30, 1984 marked the beginning of the construction phase which was completed in September of 1984.

8.2 Special Construction Requirements

One aspect of the construction project was to modify existing sanitary/stormwater regulating chambers and to construct one new in-line control chamber on an existing combined sewer line. Each of these locations continually passes dry weather sewage flows and is subject to extreme flow rates during wet weather conditions.

The Contractor performed all work without bypass of any dry weather flow to the river. Only stormwater flow conditions which previously created discharge to the river were allowed to discharge to the river during the contract work. The Contractor was required to provide all necessary means of work protection by backwatering, pumping, and bypassing around the work to the interceptor all dry weather flow and to protect all work from the effects of wet weather flow and correct any damage caused during increased flow conditions was required to prevent any material or debris required for construction or created during demolition from entering the sewage flow.

The Contractor was made aware that most regulators are below potential river levels and that he was to prevent the flow of river water into the regulator chambers, particularly during sluice gate removal and tide gate installations at Throop Street and Remington Street. Compliance with these requirements was ensured by inspection during the work.

The existing flow regulators were to be removed prior to installation of the new regulators. The Contractor removed the float-operated regulator at nine "B" type regulators including the gate, float, operator, cables, suports and appurtenances. The upstream slide gate, seat, stem, supports and appurtenances were to remain. The existing cast iron wall casting was left in place to simplify demolition and new regulator installation. At two "A" type chambers existing slide gates, seats, stems supports and appurtenances were removed and replaced with new regulators. At one "A" type regulator, Dearborn Street, the existing regulator was adjusted but not removed. One new flow regulator was installed in the new control chamber at Salt and Vermont Streets. A total of 12 new flow regulators were installed.

8.3 Installation

The new flow regulating devices and accessories were designed to simplify installation procedures. In general each unit was supplied with accessory plates designed to be attached to the existing chamber weir wall using through bolts and/or anchor bolts. Studs or threaded holes on the back plate allowed for mechanical attachment of the flow regulator. Integral anchor pads on each regulator allowed for securing the device to the chamber base slab with anchor bolts set into existing concrete or onto cast-in-place anchor assemblies where new concrete was designated. At Adams Street the flow regulator was attached to a new wall using a cast-in-place wall anchor. At the new chamber constructed at Salt/Vermont Streets, the flow regulator included an integral wall mounting plate conforming to the 42 inch wall thimble cast by the Contractor.

Flow regulators for which the overall size exceeded the dimensions of available accessways or manholes, were supplied as segmented units capable of passing through the existing openings. These devices consisted of two to three segments assembled using bolted connections. Devices were supplied assembled with the intent that the Contractor disassemble the units, place each segment into the chamber, and reassemble the unit. Intact units were then properly positioned within the chamber and mechanically secured to the wall plate assembly and the floor slab.

The flow regulator at the new regulator chamber at Salt and Vermont Streets is equipped with a 6 inch vent pipe. The vent pipe is removable from the flow regulator and segmented to allow placement in the chamber. Vent pipes are supported by attachment to the chamber wall using anchor bolts and an adjustable bracket assembly.

Three types of flow regulators were suppled including 10 conical "MC"type, one horizontal "MH" type and one orifice plate regulator. The "MC" types were used to replace existing existing float-operated regulators at all the "B" type chambers and the manual gate at the "A" type chamber at Court Street. These devices are general conical in shape with flow entering through a circular end plate. One new "MH" type flow regulator, installed at the type "A" regulator at Ames Street, consists of two flat plates connected by a band around the outer edge. The final device was a simple orifice plate consisting of a square plate with a circular hole which was installed within the Fraser Street chamber. Pertinent details of flow regulator characteristics are presented in Table 8.1 (which was included in the bid document to provide the Contractor with a means of estimating the handling involved in the installation of the devices.

8.4 Chamber Modifications

Photographs were taken to document the modification process. The photograph in Figure 8.1 displays the interior of a typical "A" type regulator chamber as previously modified under the Hancock Street Detention Facility construction program in 1977. Sewage enters the chamber via the pipe on the left and drops into a small sluice. During dry weather conditions flow then drops through a 6 inch x 6 inch opening in the side wall at the end of the sluice to a chamber section leading to the DWF outlet pipe. This pipe, located in the upper left part of the picture, leads to the interceptor drop manhole. Moderate wet weather flow, which will not pass through the 6 inch x 6 inch opening, is backed up in the chamber by the sluice end-walls and the cross wall seen to the right of the sluice. This cross wall was constructed during the previous modification program. Prior to its construction, wet weather flow would continue flowing to the right and pass through flap gates to the river.

Wet weather flow volume greater than the storage capacity of the system will pass over the sluice end walls and to the intercepter. Extreme wet weather flow rates sufficient to backwater the interceptor will cause flow in the chamber to be routed around the cross wall, back over the side wall beyond the cross wall, and through the flap gates to the river.

Although not shown in the photograph, a small manual gate was located on the back side of the sluice end-wall regulating flow through the 6 inch x 6 inch opening. As part "reduced BMP" project the sluice gate was removed and replaced with vortex flow regulating devices at two locations as detailed later.

Typical "B" type regulators are much larger than the "A" type. Figure 8.2 depicts the modified weir area and new flow regulators within a typical "B" type regulator. Basically the chambers are enlarged versions of the "A" Type with the addition of a float chamber next to the chamber outlet pipe. Flow regulation was controlled automatically by a float-operated regulator located between the sluiceway and the chamber outlet pipe. During the current program this assembly was entirely

Table 8.1 Flow Regulator Details

Regulator	* Type	OVERALL Length (in.)	Width (in.)	Height (in.)	** Approx. Wt. (lb)	*** Segmented Cone No. segments)	Diameter Intake (in)	Vented
1A Weiss	MC	74	63	53	700	---	25	Y
2 Cronk	MC	37	28	24	130	2	8	---
5 W. Genesee	MC	38	30	25	130	2	12	---
6 Remington	MC	38	30	25	130	2	8	---
8 Throop	MC	38	30	25	130	2	8	---
9 Hayes	MC	38	30	25	130	2	10	---
12 Court	MC	25	15	13	40	---	4"	---
11 Ames	MH	22	22	4	30	---	4"	---
13 Adams	MC	60	52	44	400	3	19	---
16 Mackinaw	MC	38	30	25	130	2	10	---
17 Dearborn	N	---	---	---	---	---	---	---
17A Fraser	O	18	18	0.5	40	---	---	---
18 Salt/ Vermont	MC	79	56	49	500	---	23	Y

* - Key:
MC - conical type vortex valve
MH - horizontal type vortex valve
O - orifice plate
N - no device

* excludes wt. fittings

*** see 6B-3

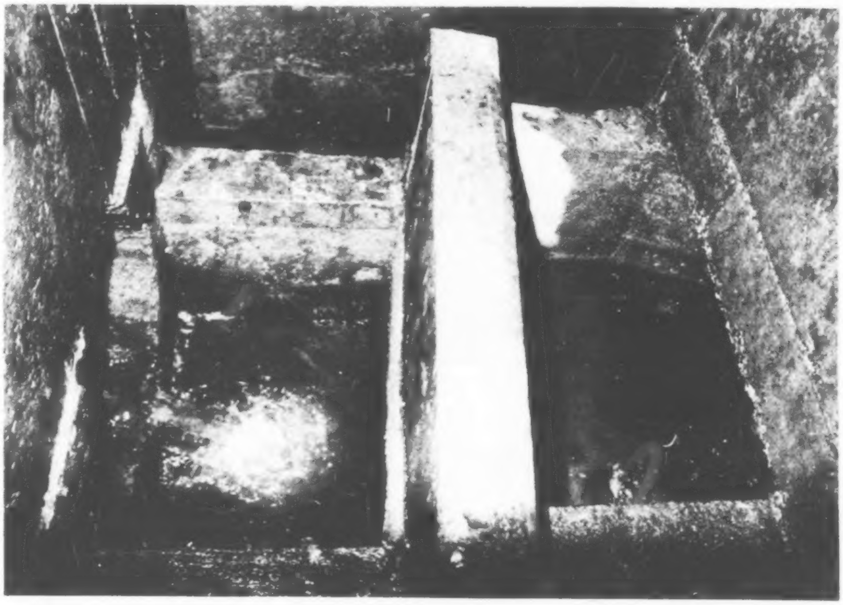


Figure 8.1 Typical Type "A" Regulator Chamber

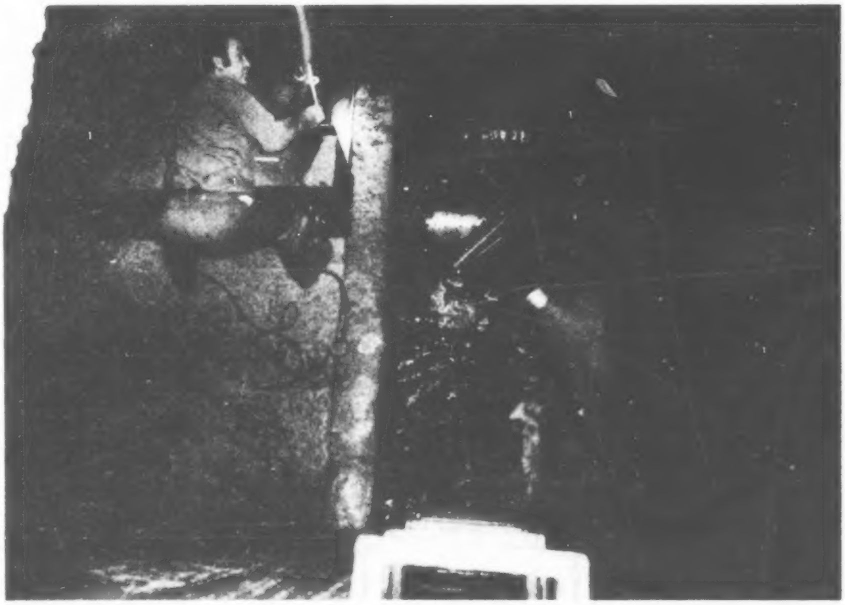


Figure 8.2 Modified Area of Typical Type "B" Chamber

removed and replaced with vortex flow regulating devices. The photograph in Figure 8.3 shows parts of a removed float-operated flow regulator.

The "B" type chambers also included manual gates on the upstream side of the side wall. Photograph A, Figure 8.4 is a view looking upstream of the sewer pipe entering the chamber, spill to the sluice and the manual gate in the sluice side-wall. Photograph B, Figure 8.4 also includes the sluice and the gate as viewed from above. Similar to the "A" type chambers most wet weather flows were originally allowed to pass through backwater gates to the river. The photograph in Figure 8.5 illustrates a typical backwater gate.

Modifications to the type "B" regulators generally included removal of the existing float-operated regulator, installation of a vortex flow regulating device, cutting down of the side overflow-walls and the construction of the central cross-wall. The photograph in Figure 8.6 displays the new cross weir and the cut-down weir on each side. Adjustable weirs were installed at four regulator locations and can be seen in this photograph.

8.5 Construction and Installation of Flow Regulators

Flow regulating devices used in this project were of variable types and dimensions as required by the degree of flow regulation and the conditions of the installation. Access to the existing regulator chambers was typically via manholes or backwater gate access holes as shown in the photograph in 8.7. In a number of instances, the sizes of the new flow regulators were much larger than the available openings such that segmentation of the device were required. Photographs A and B in Figure 8.8 and 8.9 illustrate a device dissassembled, partially assembled and completely assembled respectively. This device was installed in the Weiss Street Regulator and included a specialized inlet transition section. Photographs A and B in Figure 8.10, illustrate the device for the Salt/Vermont Street regulator chamber which incorporated an integral wall mounting flange and a removable high level stack vent. The photograph in Figure 8.11 shows the unit installed in the new chamber weir wall.

As detailed Chapter 7, the Adams Street regulator chamber vortex flow regulating device installation utilized an upstream cross wall and wall thimble. The device and detachable cast-in place wall thimbles are shown in the Photograph in Figure 8.12. The installed device and upstream cross wall, designed to direct flow to the inlet, are shown in Photograph A, Figure 8.13.

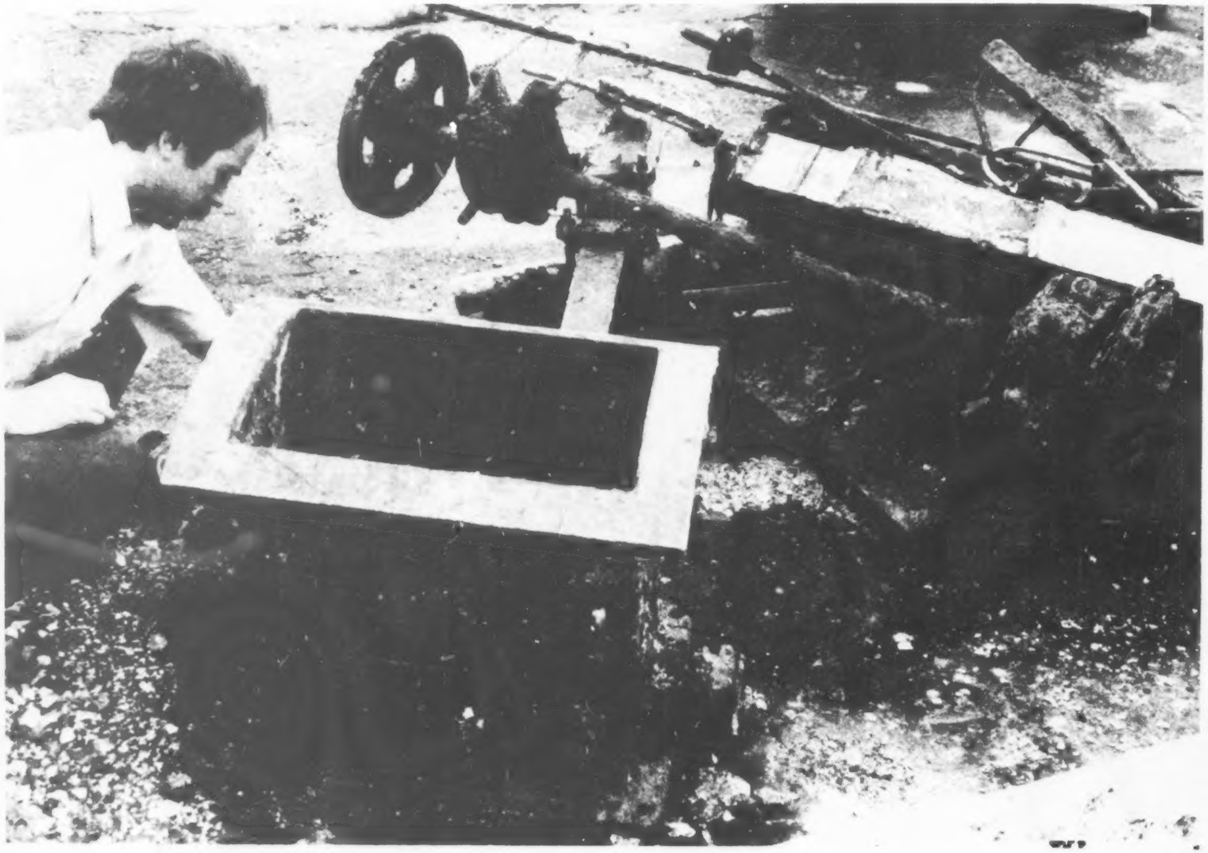
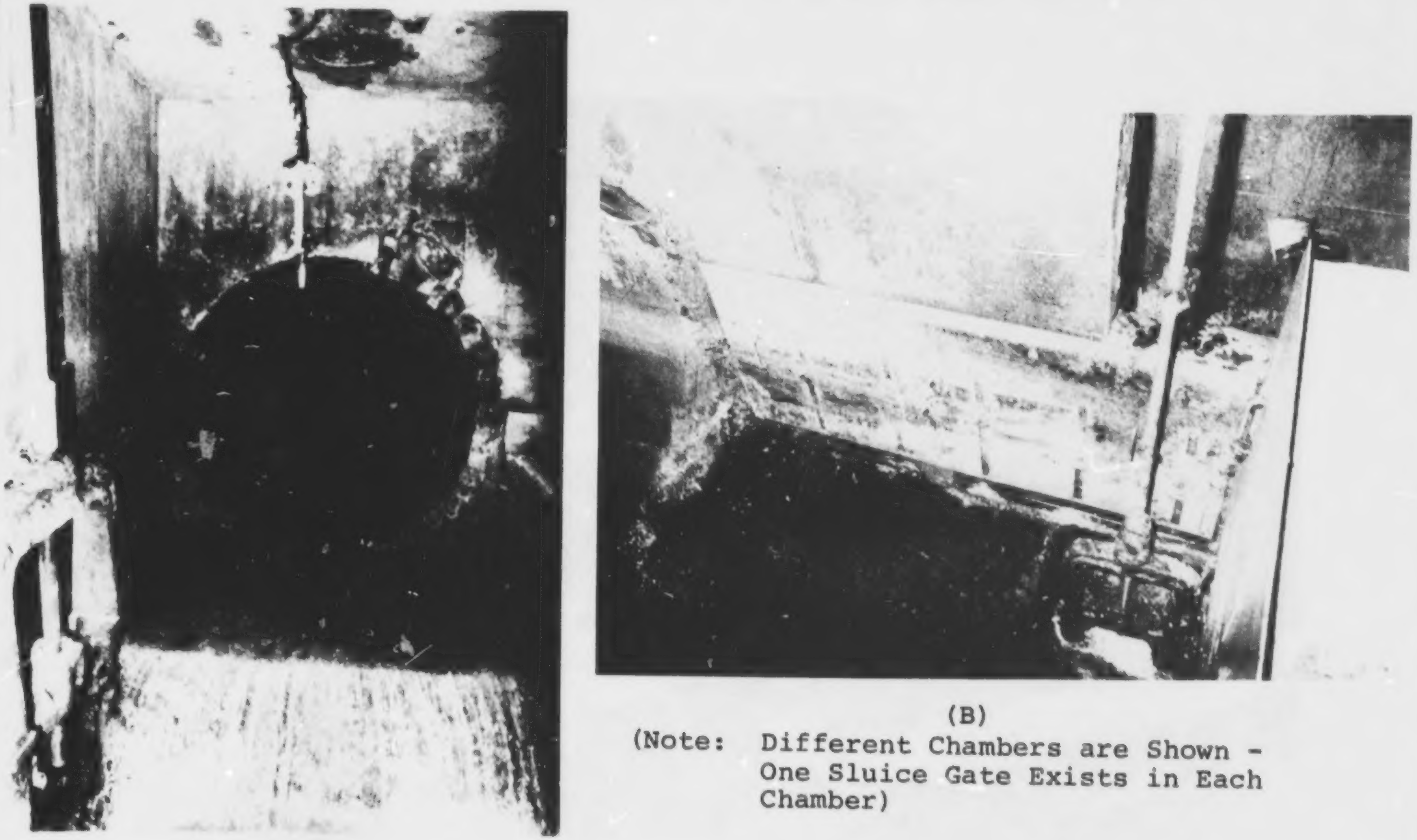


Figure 8.3 Partial Components of Brown and Brown Regulator Removed From Chamber



(A)

(B)

(Note: Different Chambers are Shown - One Sluice Gate Exists in Each Chamber)

Figure 8.4 Typical Type "B" Chamber - Influent Area

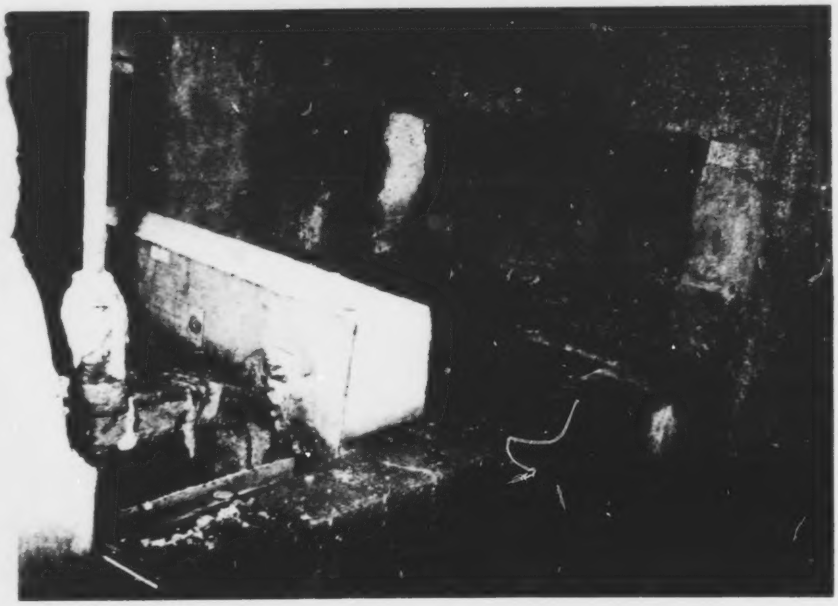


Figure 8.5 Type "B" Chamber - Flapgate to River Area

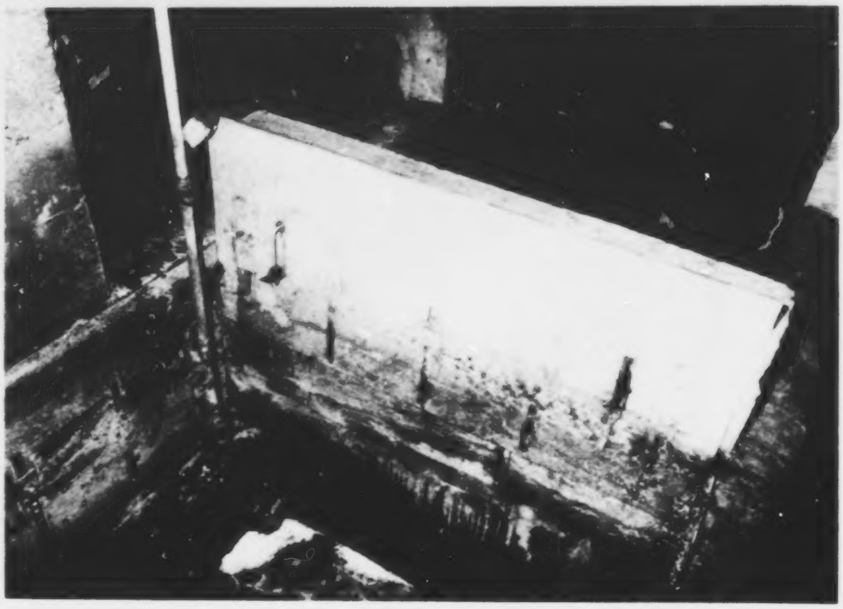


Figure 8.6 Type "B" Chamber - New Cross Weir

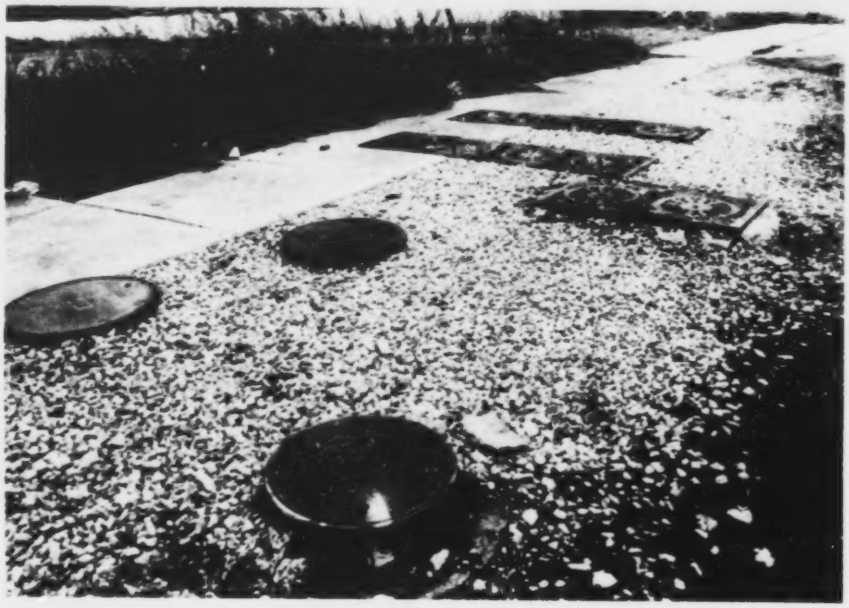
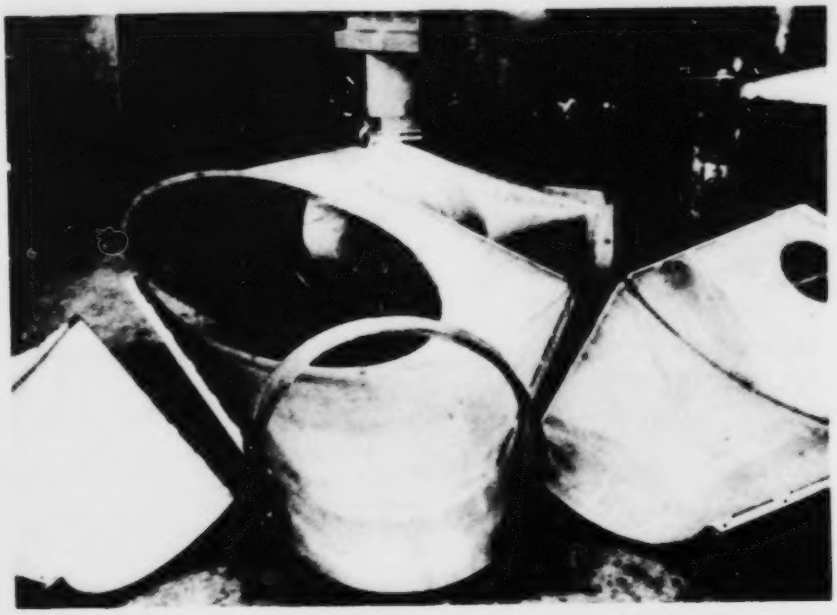
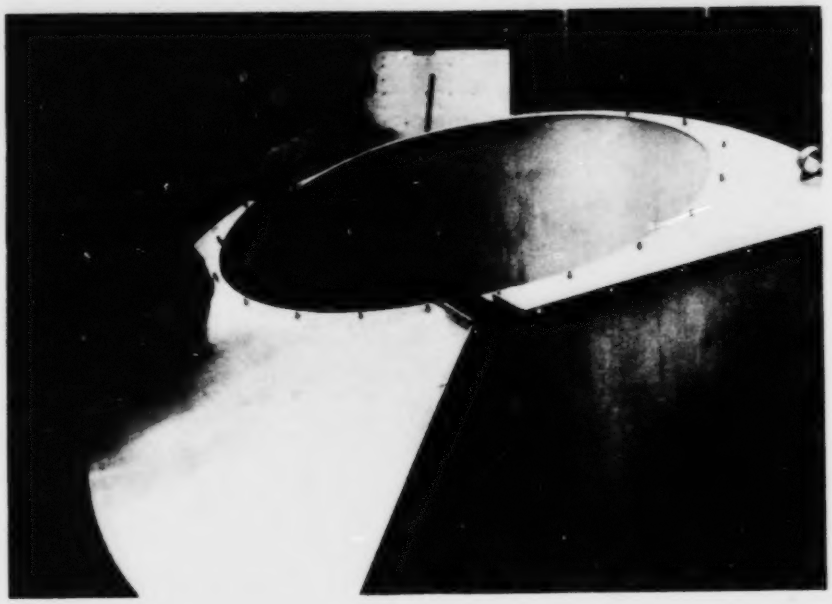


Figure 8.7 Chamber Access Manholdes and Gate Covers



(A)



(B)

Figure 8.8 Segmented Vortex Valve Regulator - Disassembled and Partially Assembled

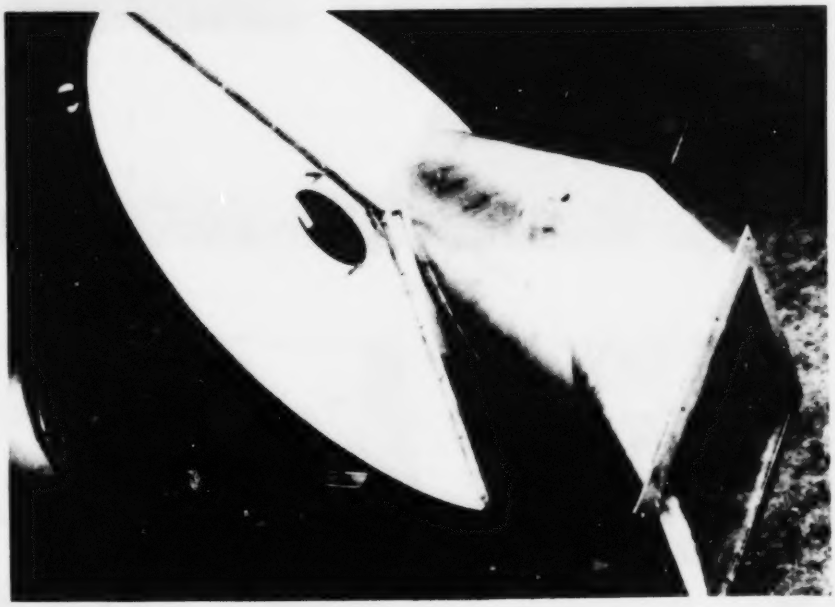
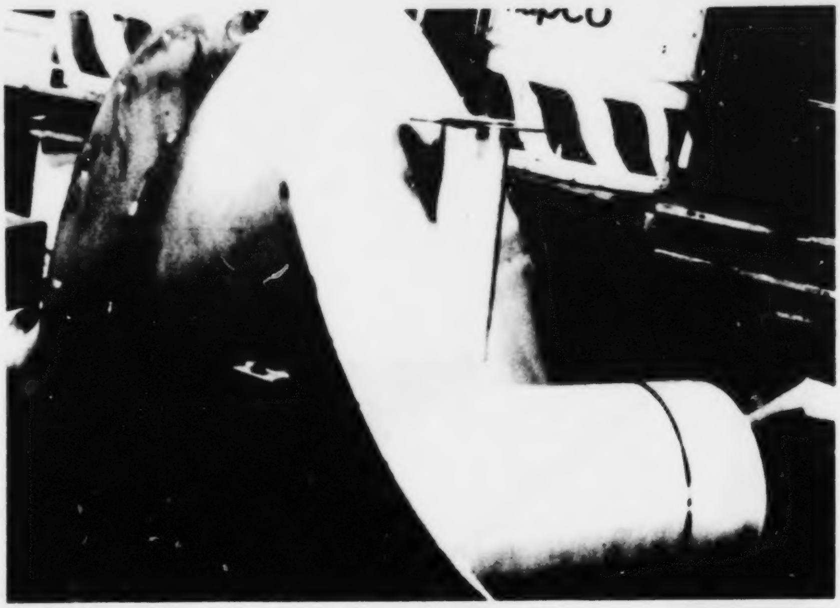
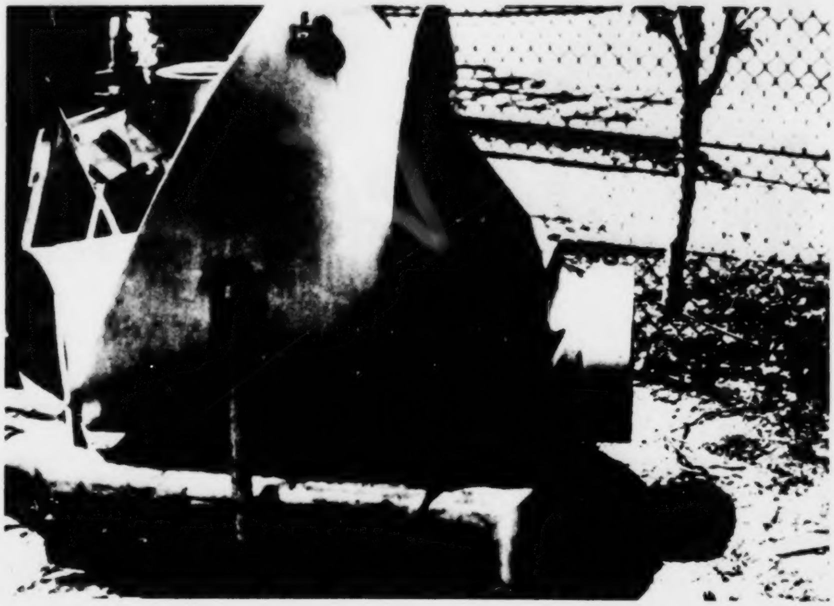


Figure 8.9 Segmented Vortex Valve Regulator - Fully Assembled
(Unit Installed at Weiss Street Chamber)



(A)



(B)

Figure 8.10 Salt/Vermont Street Chamber Vortex Valve

Figure 8.11 Installed Vortex Valve Regulator at the New Salt/Vermont Streets Storage Chamber

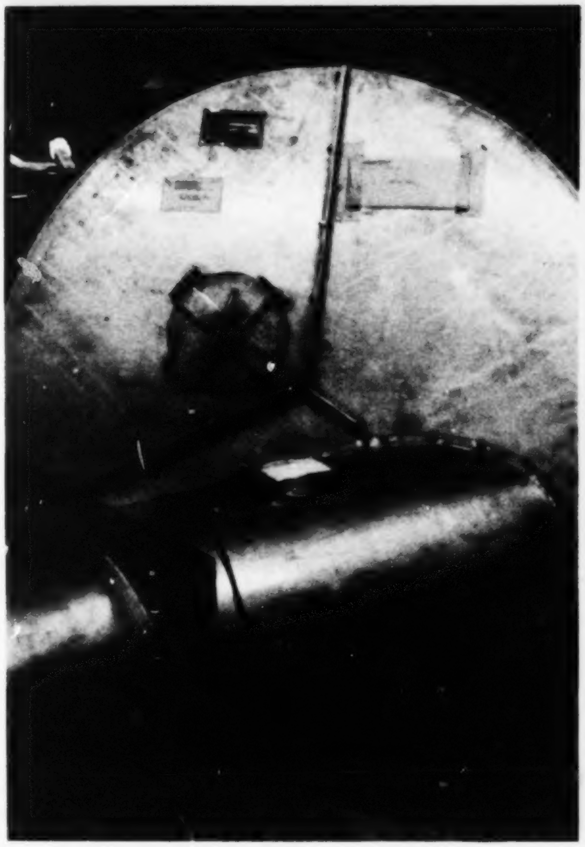


Figure 8.12 Adams Street Chamber Vortex Valve Regulator

Figure 8.13 Installed Adams Street Chamber Vortex Valve Regulator

Most of the vortex flow regulators used in this project were the conical type as shown in the Photograph in Figure 8.14. Each unit was provided with a mounting plate, designed to be permanently mounted to the wall, incorporating studs for attachment and removal of the device. For most installations, the unit's inlet sleeve was perpendicular to the wall. In several instances, the existing wall casting and the chamber outlet were positioned such that modifications were necessary. The photograph in Figure 8.15 illustrates a unit with the inlet sleeve modified by changing the sleeve angle in the wall modified to orient the device outlet to properly direct flow into the chamber outlet pipe.

A horizontal type vortex flow regulating device was deemed most suitable for the Ames Street location since the chamber outlet was located in a side wall as opposed to the standard opposite wall, limiting available space for installation and orientation. Photograph A, Figure 8.16 illustrates a typical horizontal type unit which has a side inlet and bottom outlet. On this unit the outlet is located directly below the inspection port (the cover of which is seen on the top plate). This unit was mounted flush to the wall with the plate shown and a wall bracket. Photograph B, Figure 8.16 illustrates the installed unit with the inspection port cover plate removed. The chamber outlet is located at the bottom of the wall to the left of the unit.

8.6 Weiss Street Pumping Station Wet Well Modification

The last program element of the reduced BMP program was the Weiss Street Pump Station wet well. The intent was to increase the depth of water five feet in the West Side Interceptor to maximize flow crossover the the East Side Interceptor prior to spillover into the pump station wet well. This was accomplished by construction of a spill over wall inside the wet well between the flap gates and the bar screens. The wall is shown in Photograph A, Figure 8.17 and has a crest elevation 4.75 ft. above the flap gate seat elevation. At the base of the cross wall a manual gate valve is located to allow drainage of water trapped between the walls. A three foot high wall was also constructed at the mid floor level all round the open chamber area as seen in Photograph B, Figure 8.17 to contain extreme flow levels and minimum excess splashing conditions.

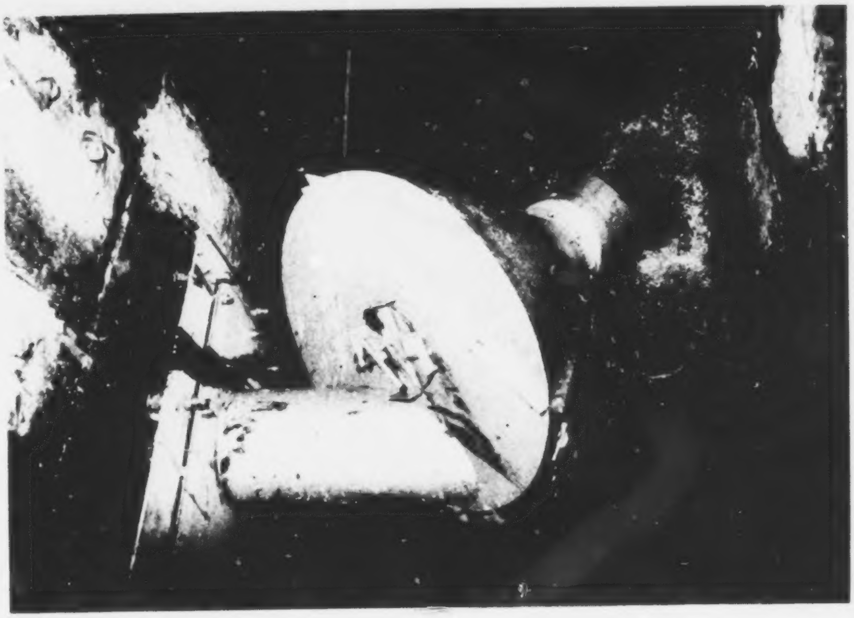


Figure 8.14 Typical Installed Conical Type Vortex Valve Regulator

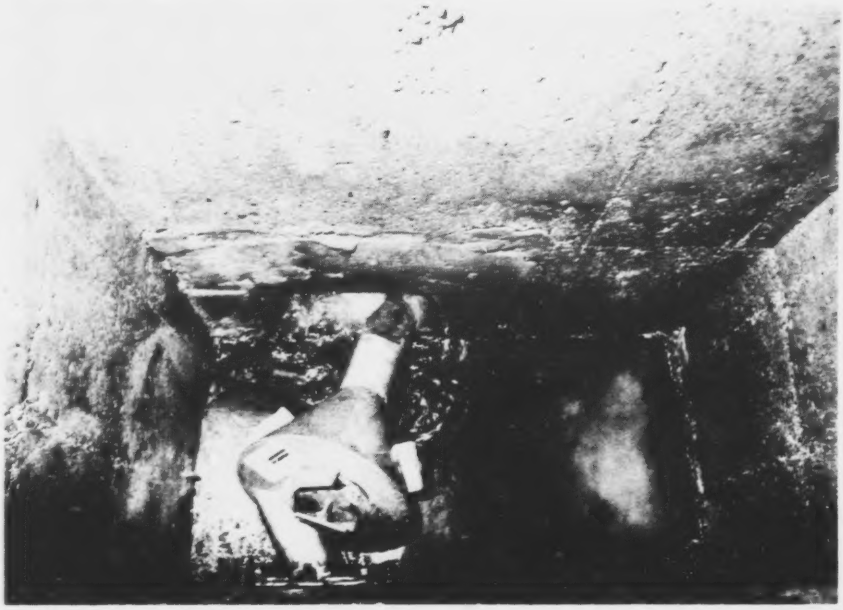
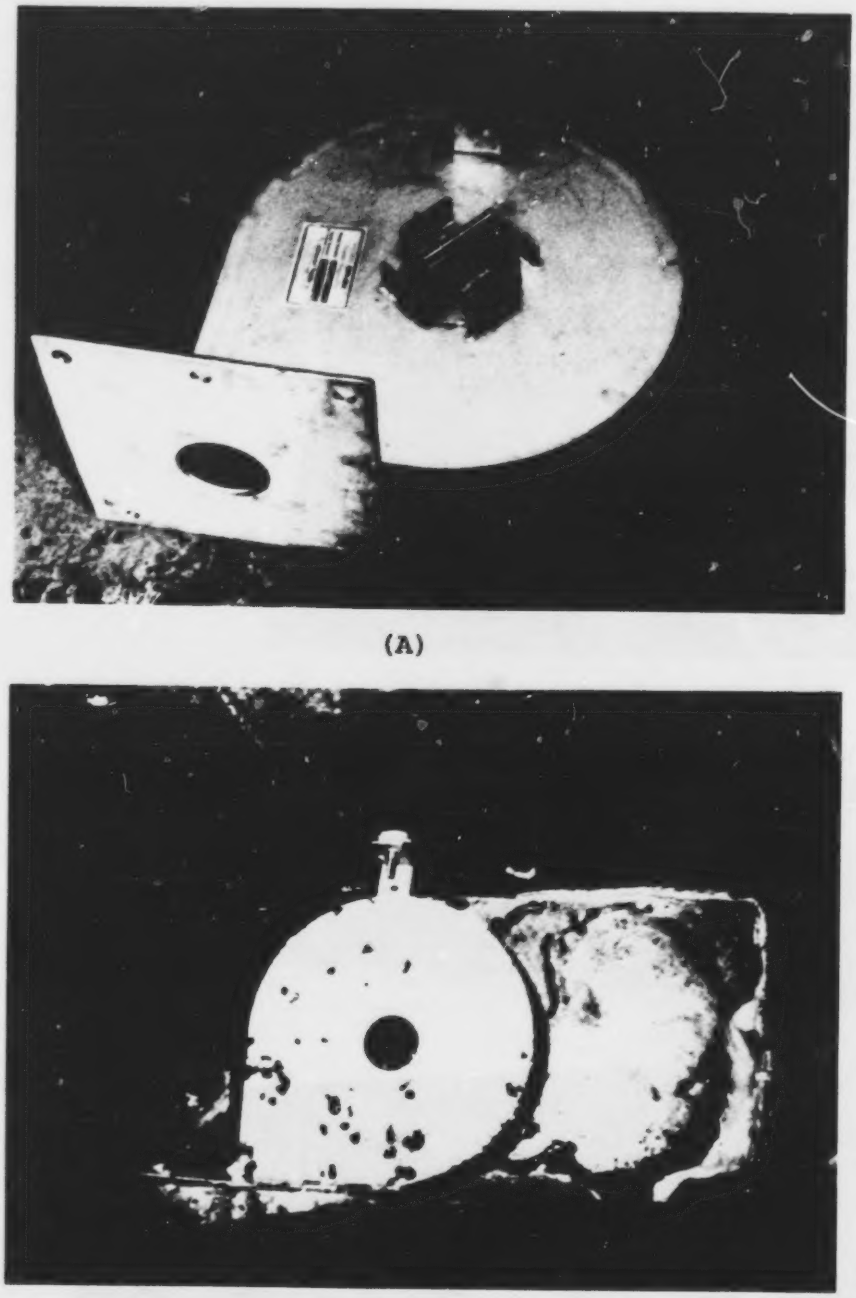


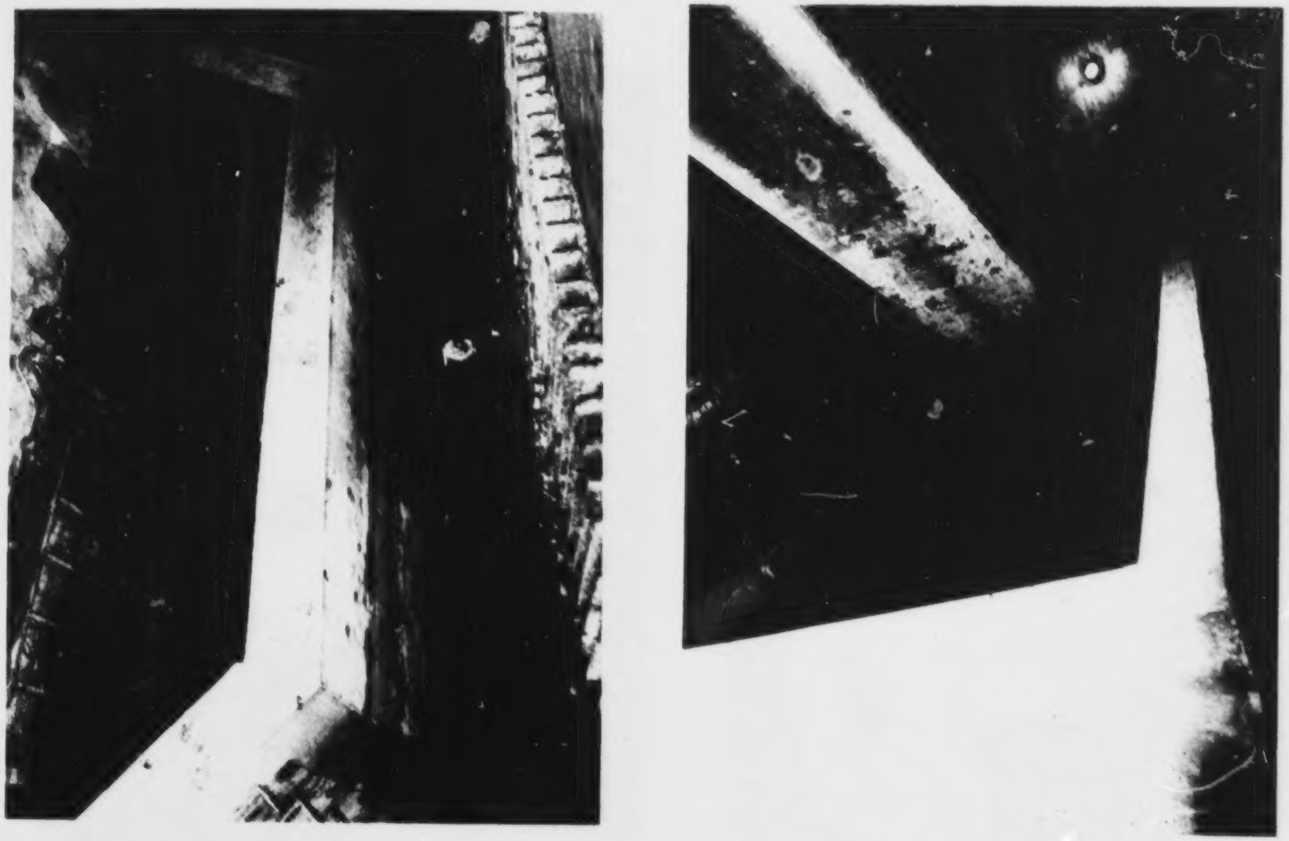
Figure 8.15 Installed Conical Type Vortex Valve Regulator with Modified Inlet Sleeve



(A)

(B)

Figure 8.16 Horizontal Type Vortex Valve Regulator



A

B

Figure 8.17 Weiss Street Pump Station — New Weir and Splash Wall

CHAPTER 9

FIELD AND MONITORING PROGRAM
POST BMP IMPLEMENTATION
DATA AND ANALYSIS

9.1 Foreword

Numerous locations were selected throughout the Saginaw drainage area during all three phases of this study for the collection of data pertinent to the generation, calibration and operation of the system simulation model and later for monitoring of the reduced BMP system modifications.

Field monitoring programs conducted for the purpose of Phase 1 and Phase 2 simulation model generation and calibration are presented in Chapter 6. Chapter 9 presents the results of the Phase 3 field monitoring program conducted after the reduced BMP program was implemented. Section 9.2 briefly describes the components of the field program. Section 9.3 summarizes the results of a program documenting backwatering effects resulting from partial BMP plan modifications. Sections 9.4 and 9.5 present data collected at the Salt/Vermont Streets chamber (new in-line storage facility) and at the Adams Street regulation chamber, respectively. Evaluation program data collected at the Weiss Street Pump Station and at the WWTP are presented in Sections 9.6 and 9.7, respectively. Field discharge measurements for several of the new vortex valves are presented in Section 9.8. Cronk Street regulator chamber data is evaluated in Section 9.9. Field observations of the modified regulators are detailed in Section 9.10.

9.2 Post Implementation Field Program

Elements of the post reduced BMP implementation field program were designed to facilitate a final review and calibration of the simulation model and to determine the practicality and effectiveness of the regulator chamber modifications in terms of in-system storage generation and the suitability of the new flow regulating devices.

Similar to earlier phases of the evaluation program, CSO rates and water quality data were recorded. In addition, the post implementation field program included river elevation determinations, in-line storage evaluation, discharge monitoring of new vortex valve flow regulating devices and documentation of maintenance requirements for the new regulating devices.

Water quality analysis included evaluating combined sewage passing to the West Side Interceptor at Adams Street and CSO to the river at this location. Water quality sampling at the Weiss Street Pump Station wet well and the WWTP grit chamber which were performed during earlier phases was also again conducted during this period.

Water surface elevations were continuously monitored at the new Salt/Vermont regulator chamber, at the Cronk Street backwater gate chamber, at two locations within the Adams Street regulator chamber and upstream of the Throop Street regulator chamber. Peak water surface elevations were monitored at manholes at five locations in the in-line storage area upstream of the new Salt/Vermont regulator chamber, at single locations upstream of two other regulator chambers (Adams Street and West Remington Street) and within three regulator chambers (Throop Street, Hayes Street and West Remington Street).

The original intent was to monitor all post construction rain events within a six week period and select the most pertinent events for complete review. Storm event measurments occurring during this period were less than ideal due to adverse conditions such as faulty backwater gate operation and mixed rain/snow precipitation, however, sufficient data was attained to reasonably assess the overall impact of the implementation program.

9.3 System Backwatering

Monitoring of the post reduced BMP plan implementation conditions included recording peak water elevations at selected regulator chambers and at locations upstream of modified regulator chambers and at the new in-line storage chamber at Salt and Vermont Streets. Peak water surface elevation determinations for wet weather periods were accomplished using cup gages constructed of 1-1/2 inch PVC pipe end caps with aluminum plate splash covers attached at equal incremental spacings to sections of electrical tubing. Gages were designed and installed by EDP to be easily removed after each event for inspection by City of Saginaw maintenance crews. Most gages were installed in manholes upstream of regulator chambers and allowed for removal from street level. Remaining gages were installed in regulator chambers and accessed from within the regulator chambers.

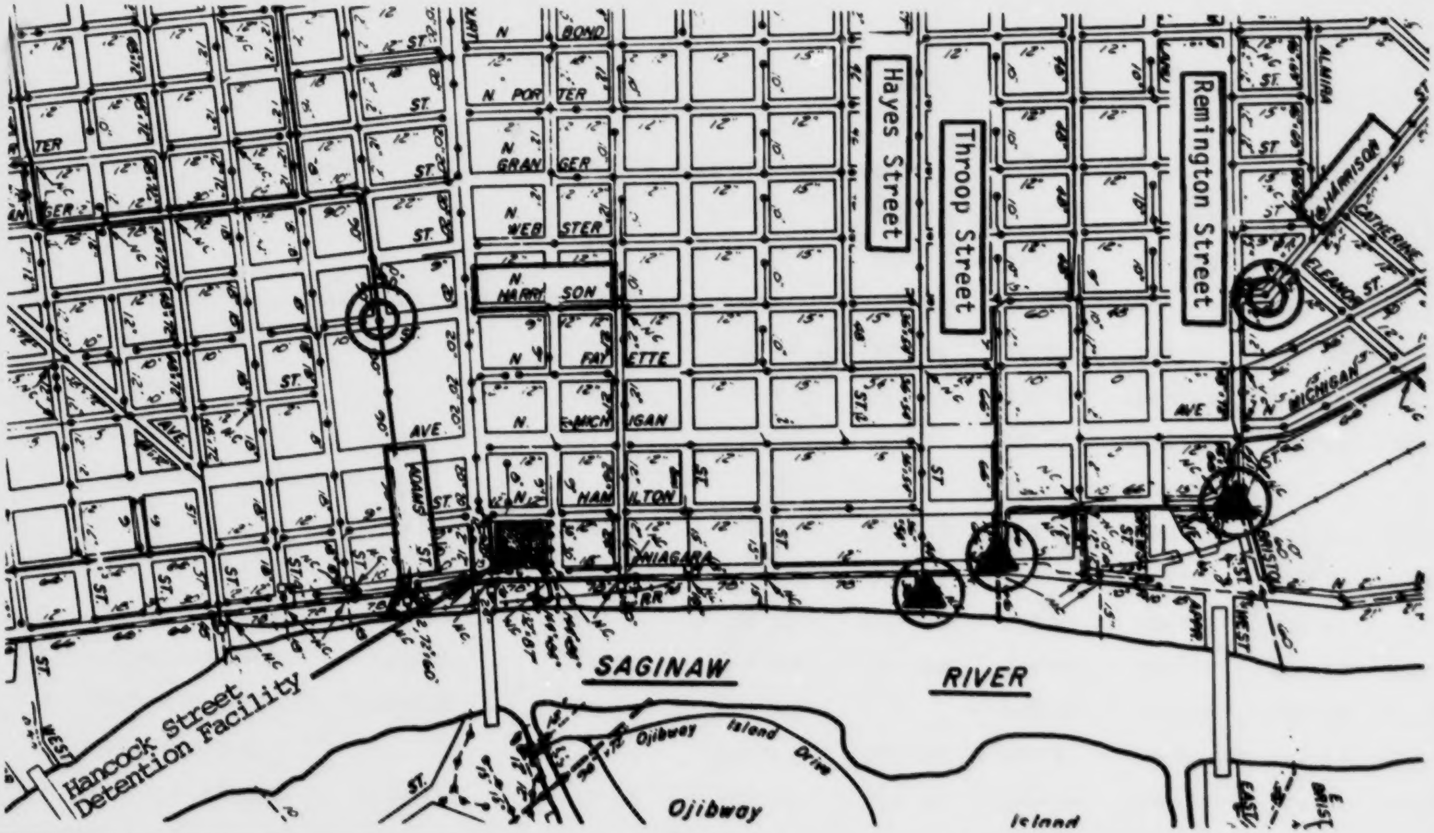
Table 9.1 includes pertinent data for the 10 cup gage locations and the downstream regulator chambers. Total rainfall accumulations for five storm events monitored are also given in Table 9.1. Locations of these measurement points are shown in Figures 9.1 and 9.2. The intent of this program was to determine

TABLE 9.1

CUP GAGES MONITORING OF PEAK WATER ELEVATIONS

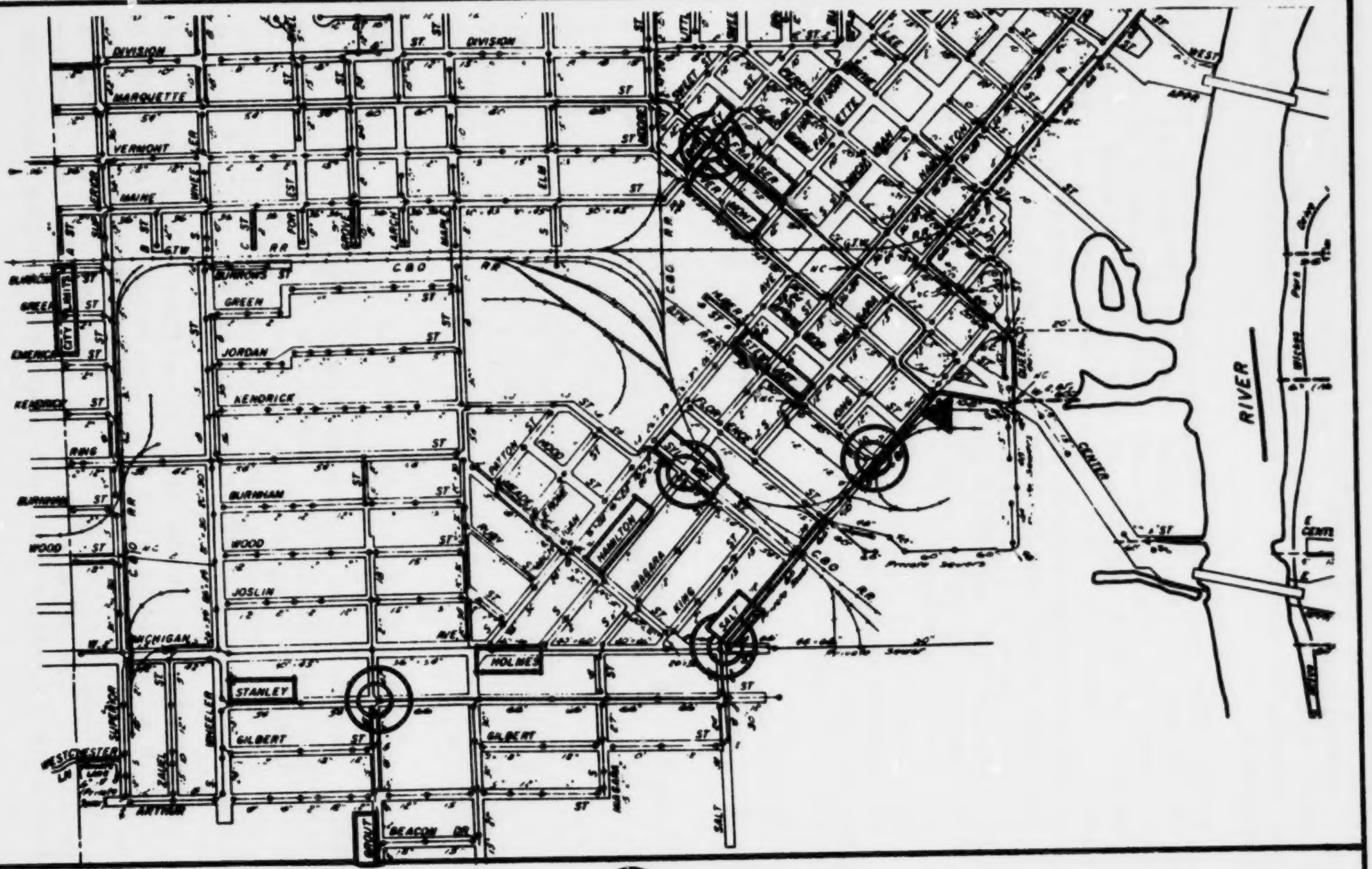
NO.	LOCATION OF MANHOLE	DOWNSTREAM REGULATOR	MANHOLE INVERT ELEVATION	REGULATOR WEIR ELEVATION	1984 STORM DATES/PEAK ELEVATIONS				
					10/18	10/21	10/26	11/1	11/9
1	GROUT & STANLEY	FRASER	582.72	584.42	584.30	<584.30	<584.30	<584.30	584.30
2	SALT & HOLMES	FRASER	579.64	584.42	583.64	584.64	583.64	584.64	585.64
3	SYLVAN & HAMILTON	FRASER	582.74	584.[illegible]	*	>589.00	584.00	585.00	>589.00
4	STEWART & SALT	FRASER	577.21	584.42	584.88	584.88	585.88	584.88	582.88
5	FRASER & BRALEY	FRASER	582.30	584.42	584.63	584.63	584.63	588.63	*
6	ADAMS & HARRISON	ADAMS	580.22	581.50	>587.39	>587.39	581.89	>587.39	>587.39
7	HARRISON	W. REMINGTON	584.42	585.45	587.50	587.00	586.00	587.50	587.50
	LOCATION AT REGULATOR								
8	HAYES		583.30	584.97	585.13	585.63	<584.13	585.13	585.63
9	THROOP		579.69	582.60	583.52	583.02	<580.52	583.52	583.52
10	REMINGTON		582.95	585.45	586.28	585.28	586.28	586.28	585.28
STORM EVENT PEAK ELEVATION RECORDED AT REGULATOR				FRASER:	584.40	584.65	577.18	584.90	584.40
				ADAMS:	582.30	582.44	581.34	582.85	582.85
RAIN ACCUMULATION (INCHES)					0.75	0.79	0.17	0.61	0.73

* ALL CUPS PARTIALLY FULL - INDICATES POSSIBLE SPLASHING EFFECT FROM CATCHBASIN LEADER PIPE DISCHARGE


Hayes Street
Throop Street
Remington Street
Hancock Street Detention Facility
SAGINAW
RIVER
Ojibway Island Drive
Ojibway
Island
N BOND
N PORTER
N GRANGER
N WEBSTER
N HARRISON
N FAYETTE
N MICHIGAN
N HAMILTON
NIAGARA
ADAMS
HARRISON
ELEANOR
CATHERINE
MICHIGAN
ALMIRA
BRISTOL
APPR.
WEST
EAST
AVE
ST.

LEGEND: ▲ Modified Chamber ◎ Cup Gage Location

Figure 9.1 Hancock Street Area Cup Gage Locations


RIVER
DIVISION
MARQUETTE
VERMONT
MAINE
C B O RR
GREEN
JORDAN
KENDRICK
RING
BURNHAM
WOOD
JOSLIN
MICHIGAN
STANLEY
GILBERT
HOLMES
HAMILTON
KING
SALT
CENTER
WEST
SUPERIOR
WHEELER
ARTHUR
BEACON DR
CITY LIMITS

LEGEND: ▲ New Chamber ◎ Cupgage Location

Figure 9.2 Salt/Vermont Streets Area Cup Gage Location

if any adverse conditions developed due to implemented modifications. Undesireable levels of stormwater backup would be recorded as peak upstream water elevations sufficiently above levels in the regulator chamber to indicate backwater-induced surcharge conditions.

The first five cup gage locations are upstream of the new Salt/Vermont Street in-line storage chamber (Table 9.1). At four of these locations peak water elevations indicate that no surcharge occurred in that portion of the sewer system during storm conditions. Data at the Sylvan and Hamilton Street location indicated surcharge conditions for two events. It is possible that cup gage readings at this location were influenced by splashing effects from a catchbasin leader pipe entering high in the manhole, since for one additional event most cups were noted to be only partially full. No basement floodings had ever been noted in this drainage area prior to the chamber construction nor were any noted during the post construction monitoring. If surcharging had occurred, depths were limited to less than problem levels which is quite possible since the 54 inch pipe serving the area is 20 ft. below ground level.

No indication of surcharge conditions was evidenced at the Harrison Street location upstream of the West Remington Street regulator chamber.

Although four of five events indicated high water conditions at the Adams Street and Harrison Street manhole upstream of the Adams Street regulator, available data indicates that the new weir with a crest elevation of 3.3 ft. above the 7.5 ft. pipe invert does not induce backwatering. Based on calculation of the pipe capacity at the upstream manhole as 253 cfs, and the estimated hydraulic profile and net flow capacity over the regulator chamber spillover weir at the recorded depth, no restriction was evidenced.

Hydraulic limitations of the connector pipe from the Adams regulator chamber to the West Side Interceptor sewer have been calculated to cause flow bypass to the river at storm flows less than the inlet sewer capacity. Overflow to the river would result and was evidenced during several of the monitored events. Backwatering due to the connector pipe hydrualic restriction at the Adams Street regulator is sufficient to cause spillover to the river but not to surcharge the inlet sewer. Flap gates allowing flow to the river provide near free-flow conditions and no excessive depths were noted with the continuous level sensors within the chamber. High water depths within the sewer line at the cup gage monitoring station appear to be due to the limited hydrualic capacity of the sewer in relation to peak flow conditions. No basement flooding has been reported in the drainage area either before or after the modification work. As with the Fraser Street regulator tributary drainage area, the sewer trunk lines are more than 20 ft. deep and surcharging of the line would not necessarily create basement flooding conditions.

At the final three locations (Hayes Street, Throop Street and Remington Avenue regulators) cup gage data from within the three regulator chambers did not indicate excessive water levels. Depths were generally less than one foot over the chamber weirs such that no sewer surcharge or spillover to the river occurred. Under the orignal design it was intended that excess flows above the allotted rate through the new regulating device spill to the interceptor. Under extreme conditions, where the interceptor capacity was exceeded flow would pass both over the new cross weir and back over the side weirs to the river. Monitored events indicated that no flows have yet passed over the cross weir which was as expected considering the mild intensity of the events which occurred. Field data collected to date indicate that the cross weir elevations selected during the design phase have been adequate to prevent premature flow to the river.

In sum, no adverse hydraulic profiles were noted at any of the locations during the post reduced BMP implementation evaluation program.

9.4 Evaluation of Effectiveness of New In-line Storage Chamber Constructed at Salt and Vermont Streets

Prior to the reduced BMP plan implementation, the Fraser Street regulator chamber controlled flows from a 441 acre area in the southwest part of the city. Location of a new upstream storage/regulator chamber was selected to be 450 ft. upstream of the existing Fraser Street chamber at the intersection of Salt and Vermont Streets (Figure 9.2). See Chapters 7 and 8 for design and implementation details.

Monitoring of wet weather flow depths at the Fraser Street regulator was performed during the Phase 2 field evaluation period of 1983. Data accummulated are representative of conditions existing prior to implementation of the BMP Plan modifications in this area. After Phase 3 construction in 1984, monitoring of stormwater depths was performed at the new chamber. The difference in sewer invert elevations between the two chambers is negligible for comparative hydraulic analysis of pre/post conditions.

Figure 9.3 illustrates the rain accumulation data for storms occuring on 8/1/83 and 10/18/84. The 8/1/84 and 10/18/84

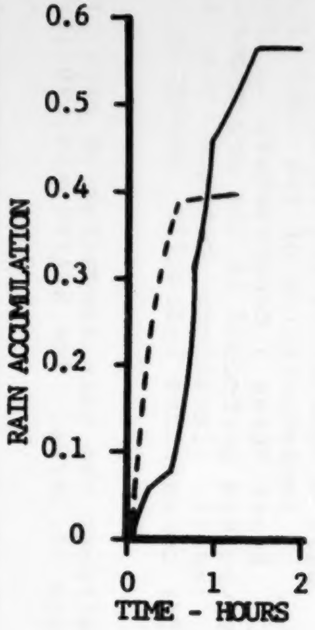
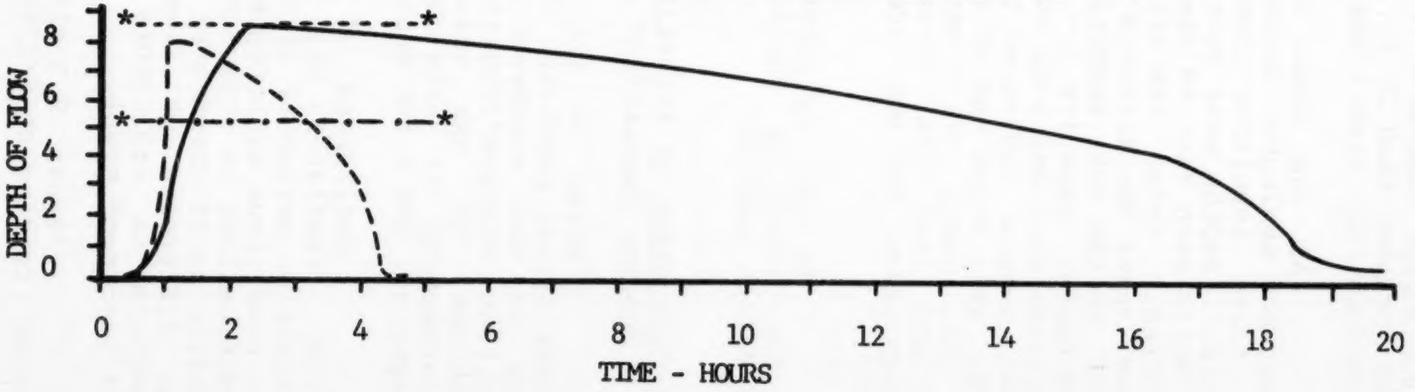


FIGURE 9.3- RAIN ACCUMULATION VS TIME

FIGURE 9.4 SEWER FLOW DEPTH VS TIME



events were selected for detailed hydraulic evaluation since they are representative of the hydraulic conditions typical of their respective pre/post reduced BMP plan implementation period and are generally similar in terms of rain intensity, duration and total accumulation.

Sewer flow depths shown in Figure 9.4 roughly indicate the rainfall to runoff time relationships. The shape of the 8/1/83 event combined sewage depth of flow curve is typical of all pre-BMP Plan events observed. The rate of depth increases varied among events dependent upon the rain condition, but all events displayed the rapid rate of depth decrease at storm end. Initially, the backwater gates allowed rapid discharge of combined sewage to the river down to a sewer depth of just over half the 9 ft. pipe diameter. Thereafter, the existing float-operated regulator controlled the outflow rate.

The shape of the 10/18/84 sewer flow depth curve is typical of all post-BMP plan implementation events. Depths of flow during the initial storage volume fill period varied as before (correlating with storm intensities), but always resulted in long-term drain down periods.

Since only rainfall and water depths within the chamber were monitored, calculations were performed using available data and the manufacturer's rating for the flow control device to determine the degree of inflow/outflow/storage balance. This procedure is described below:

Figure 9.5, which indicates the volume of stormwater storage throughout the 10/18/84 event, was prepared using the depths recorded in the field (Figure 9.4) and the depth to storage volume relationship developed during Phase 1 and Phase 2 studies. Using the storage volume curve of Figure 9.5, the rate of change in storage volume was estimated throughout the event since it is a direct function of the inflow rate, the outflow rate through the regulating device and spillover, if any. For this event total rain accumulation was recorded at the Emerson Street Station and at the City Water Works Building as 0.57 and 0.67 inches respectively, averaging 0.62 inch. Considering the land use (and appropriate runoff factor), a total stormwater volume of 397,000 cu.ft would be expected. Dry weather flow for the total rain and system draindown period of 18 hours was estimated to equal 72,600 cu.ft. using dwf rates established during Phase 1 and 2 measurement programs.

The rate of inflow (Figure 9.6) at any time was calculated as the rate of volume change plus the rate of outflow. For periods of no spillover, outflow was limited to rates controlled by the new vortex valve regulator. Using the device's

FIGURE 9.5 VOLUME OF STORED COMBINED SEWAGE VS TIME

10/18/84 EVENT POST BMP PLAN IMPLEMENTATION

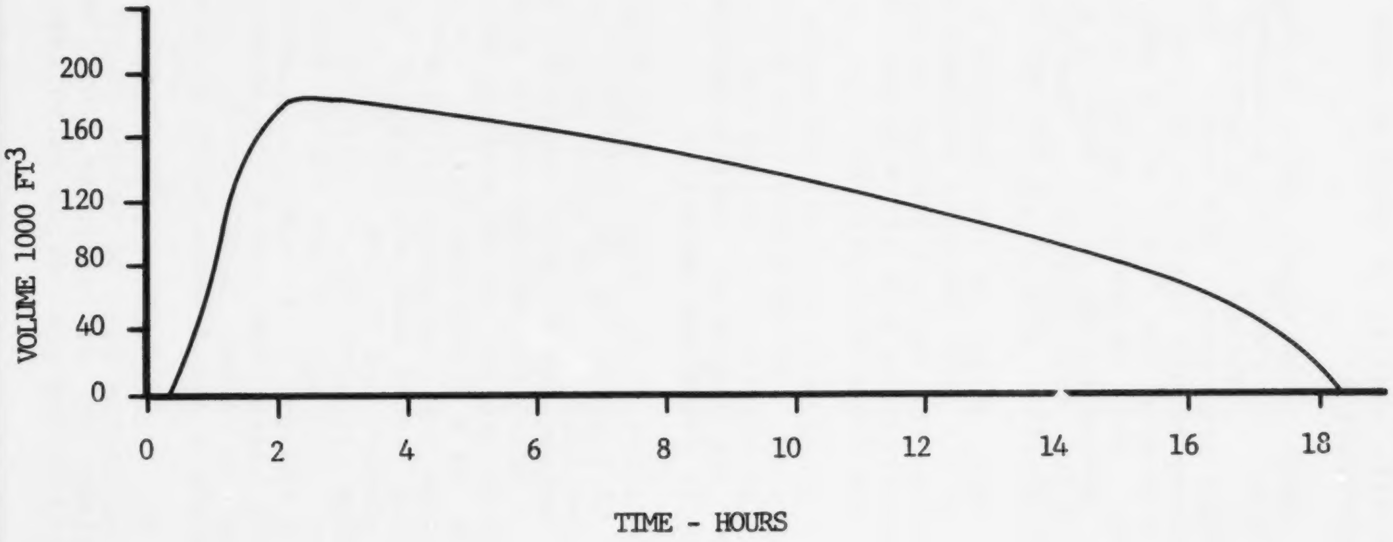
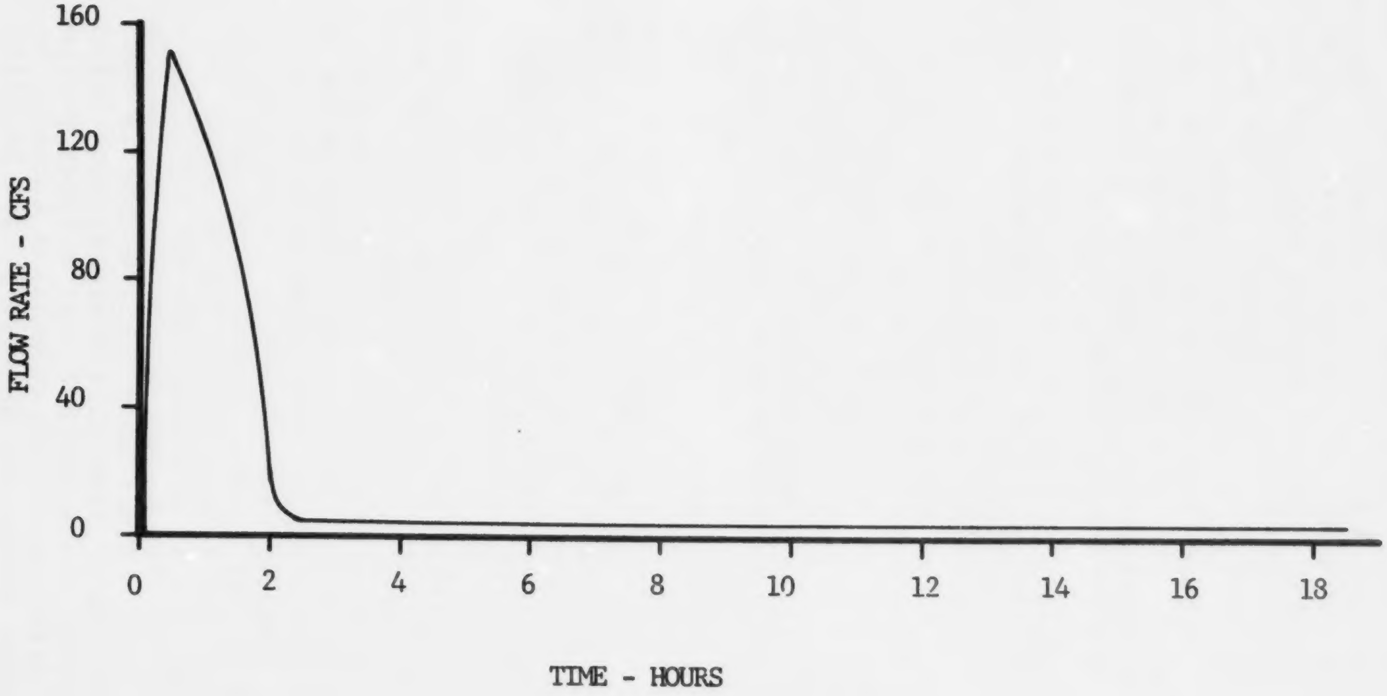


FIGURE 9.6 COMBINED SEWAGE INFLOW RATE VS TIME

10/18/84 EVENT POST BMP PLAN IMPLEMENTATION



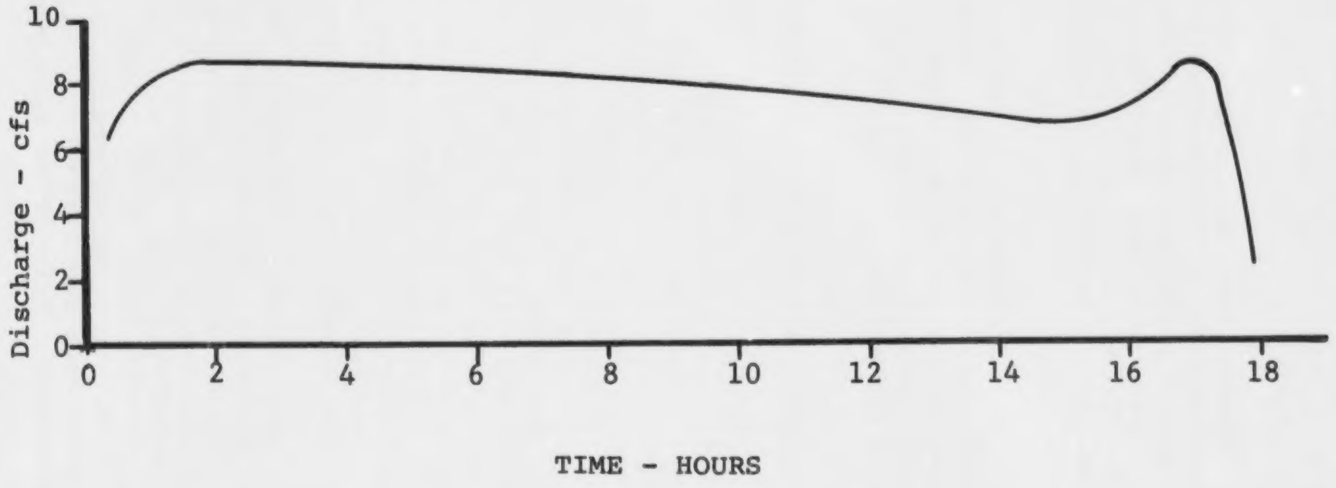
stage/discharge characteristic data (See Chapters 5 & 7) and sewer flow depth indicated in Figure 9.4, the outflow rates depicted in Figure 9.7 were calculated. The curve illustrates the unique ability of the vortex valve throttling device to maintain a near constant outflow rate with change in depth and the characteristic final high flow through rate which is intended to promote scouring. In addition to the curve values shown in Figure 9.7, inflow would also include the volume of spillover, if any. Figure 9.4 indicates that the chamber water depth remained at the weir level for a very short period of time at the 2 hour duration point of the event implying that little or no overflow would have occurred.

Integration of the vortex valve regulator device outflow rate curve (Figure 9.7) over the event duration results in a total estimated outflow of 454,000 cu. ft. This value is 15,000 cu. ft. less than the estimated combined dry and wet weather flows noted above and represents either the overflow volume, the error in values estimated or both. This difference represents 3% of the total storm flow and since minimal spillover occurred, it is concluded that the flow analysis is reasonably accurate. This implies that the vortex valve throttling device operates properly according to the manufacturer's design curve, the depth to storage volume relationship developed for the pipe system is reasonable and the runoff coefficient and land size are accurately estimated.

A similar evaluation of the pre BMP conditions is difficult due to the unknown and variable flow rates through the existing float-operated regulator and the backwater flap gate to the river. For comparative purposes of pre and post BMP conditions, rough estimates of hydraulic conditions were made for the 8/1/83 event as follows.

Total rain accumulation for this event was recorded at the Emerson Street Station and at the City Water Works Building as 0.43 and 0.49 inches, respectively, yielding an estimated runoff volume of 292,000 cu. ft. Dry weather flow for the 4 hour event totalled 16,000 cu. ft. Considering the unknown outflow rates, storage volume fill rates were not used to estimate inflow rates. Combined sewage depths in the chamber above 5 ft. cause overflow to the river. Depths above this level occurred for over 2 hours of the 4 hour event. The in-system storage volume of the 87,000 cu. ft. is available and the average outflow rate from the regulator for the 1 hour drain-down period recorded was estimated as 24 cfs. Since this outflow rate is much greater than the allowable rate from this drainage area based on WWTP capacity constraints, it is assumed that a large portion of the flow would either overflow directly at this location or be pumped to the river from either the Hancock Street facility or from the Weiss

Figure 9.7 Vortex Valve Discharge Rate vs Time



Street Pumping Station. Using the two extremes of: 1) only flow through the backwater gates spills to the river or; 2) all flow above the allotted volumes based on WWTP capacity either flows or is pumped to the river, CSO discharges are estimated to range from 124,000 cu. ft. up to 178,000 cu.ft. For the 8/1/83 pre-BMP event, 40% to 58% of the combined sewage inflow is thus estimated to discharge to the river.

Comparison of the two pre/post-BMP storm events shows the effectiveness of the in-line storage concept. As stated, a typical post implementation event (10/18/84) with 40% more rainfall than the pre implementation event, resulted in a discharge of at most 3% of the incoming combined sewage in comparison to an estimated overflow of 40-58% under pre-project conditions.

In conclusion, implementation of the BMP plan recommended upstream storage facility at this location has substantially reduced pollutant loading to the river by dissipating peak flow conditions in the system and allowing for long draindown periods to the WWTP.

9.5 Monitoring and Evaluation at the Adams Street Regulator

Intensive monitoring efforts were performed at the Adams Street regulator chamber as part of the overall field evaluation. During the pre BMP period a level sensor was installed to monitor water elevations within the dry/wet weather flow channel of the regulator chamber upstream of the float-operated regulating device. No flow monitoring was performed during the reduced BMP construction phase in the chamber. After modifications were completed, sensors were placed to monitor water levels both in the dry/wet weather flow channel upstream of the new vortex valve flow regulator and at the downstream end of the side weir to monitor spillover of extreme event flow to the river. In addition, automatic sampling devices were installed at each location, activated by flow elevation, and intended to produce representative samples of total flow through the regulator and of spill to the river.

The control concepts used during the design phase included assuming "first flush" effects within the combined sewage. This effect is the result of initial flow volumes during storm events incorporating high concentrations of street washoff materials and scouring sediment in the sewer lines. Scouring should occur early in the event followed by relatively cleaner flow representing sewage diluted with clear stormwater. At this stage two scenarios may occur. Under low rate stormwater flow conditions, velocities within the sewer may be conducive to settling of solids materials. High flow rates may produce velocities negating any deposition effects. After the end of a storm event and near the end of the storage pool drain-down period, an increase in flow rate. designed as part of the operating characteristic of the vortex valve regulating device, should occur. It is expected that if settling of solids materials occurred in the backwatered stormwater, then the final flow increase would create a second scouring phase and increased pollutant concentrations.

Samples were collected from within the Adams Street dry/wet weather flow channel on four occasions and at the weir spilling over to the river on one occasion. Samples collected at half hour intervals were composited to represent the "first flush", "mid-event" and "drain-down" (post storm) periods and analyzed for SS, BOD and total/soluble phosphorous. In addition, discrete sample analysis was performed on occasion, to determine time-variant solids concentrations.

Table 9.2 summarizes the data generated. For each event, concentrations of the four pollutant parameters are indicated based on the three storm periods. Discrete samples (if any) are then listed giving solids concentration and time of collection. As a rough comparison of peak storm flows, depths in inches over the side weir to the regulator and over the cross weir to the river, are included in Table 9.2. It should be noted that the highest peak flow over the upstream weir is not necessarily the most likely event to cause overflow to the river. It is mainly the interceptor capacity that governs if surcharge and overflow to the river will occur. Rain data for each event including total accumulation and peak hourly intensity are also recorded in Table 9.2.

Monthly average influent WWTP concentrations of suspended solids and BOD were 71 mg/l solids and 55 mg/l BOD in October and 72 mg/l solids and 51 mg/l BOD in November, 1984. Data for the first three events clearly includes initial storm samples of flow passing to the interceptor with concentrations much higher than WWTP averages confirming the "first flush" effects. "Mid-storm" sample concentrations decrease substantially but are still in the range of dry weather flow values. Dilution of dry weather flow alone with clean stormwater would produce lower concentrations such that the "mid storm" values recorded indicate continued effects of scouring and street washoff. Considering that the pipe volume upstream of the regulator is backwatered, it could be expected that not all scoured material would immediately flow to the regulator. "Post storm" samples are consistently less concentrated than "mid-storm" samples and monthly average dry weather flow indicating a condition of sewage diluted with relatively clean

TABLE 9.2

ADAMS ST. REGULATOR CHAMBER STORM SAMPLES
Concentrations mg/l

DATE	PARAMETER	START STORM	MID STORM REG	RIVER)	POST STORM	PEAK FLOW DEPTH (INCHES) OVER WEIR REGULATOR SIDE	RIVER SIDE	RAIN DATA ACCUMULATION INCHES	PEAK INTENSITY IN/HR.
10/21	SS	105	73		30	12.	4.	0.79	0.20
	TP	1.59	.83	(.25)	.89				
	SP	.85	-	(.21)	.73				
	BOD	67	27		25				
11/1	SS	424	54		10	17.		0.61	0.32
	TP	3.88	.54		.55				
	SP	.77	.28		.52				
	BOD	258	51		21				
DISCRETE									
1	SS	60	Midstorm: at max rate of depth decrease						
2	SS	7	Poststorm: at predicted flush depth						
3	SS	40	After post storm: 28 hrs after #2						
4	SS	60	" " " 29 " " "						
5	SS	34	" " " 30 " " "						
6	SS	30	" " " 31 " " "						
11/9	SS	428	77		21	18.		0.33	0.09
	TP	3.81	1.04		.46				
	SP	1.33	.59		.35				
	BOD	138	68		19				
DISCRETE									
1	SS	38	Midstorm: Just after peak conditions						
2	SS	81	Midstorm: During end of stored stormwater drain down						
3	SS	176	Midstorm: Second flow peak level condition						
11/11	SS		21		48	0.		0.38	0.09

Code: SS - Suspended Solids
TP - Total Phosphorous
SP - Soluble Phosphorous
BOD - Biochemical Oxygen Demand

stormwater possibly in combination with materials settling effects. The significance from a pollution reduction standpoint is that the initial highly concentrated flow would pass to the interceptor and later overflow, if any, would be far less concentrated.

Visual observations of final drain-down periods were recorded by EDP personnel during flow calibration of the installed vortex valve regulating devices. A short period of noticably concentrated material was visually noted ("black" in color) discharging from the device during the draindown period following a rainstorm occurring in early October. Unfortunately, the flow calibration crew was not setup for sampling at that instant. The field sampling program thereafter was tailored to analytically verify "post storm" scouring.

Discrete stormwater samples were visually inspected to determine if any highly concentrated conditions occurred. Samples with a darker appearance were tested individually for solids concentration. If no increase was visually apparent, samples for discrete analysis were selected based on depth of flow data to determine predicted peak flow and therefore potential scouring conditions.

For the 11/1/84 event the first discrete sample (Table 9.2) represented "mid storm" conditions during the period of maximum depth decrease rate. The resulting solids concentration value of 60 mg/l was close to the average storm period value of 54 mg/l indicating no particular flushing effect at that time. The second discrete sample was selected from the "post storm" period and also indicated no flushing effect. The final four samples were taken over 28 hours after the flow rate stablized at just above DWF levels. Solids concentrations are high but data is inconclusive as to whether this is a flushing effect or normal DWF concentrations.

The 11/9/84 event consisted of two intense rainfall periods and two distinct water storage periods. Three discrete sample periods were selected for solids analysis. Discrete samples from the first two periods indicated solids concentrations similar to "mid storm" composite conditions. Analysis of the last discrete sample, collected at peak storage conditions during the second storage period indicated a definite increase in the solids concentrations potentially due to the combined effects of additional street washoff and scouring of materials settled during the initial drain down period. No samples collected during the second drain down period were noticeably polluted and no analyses were run to investigate flushing effects.

In sum, "first flush" effects were clearly indicated by the field data for three complete storm events monitored justifying the improvement plan concept of modifying regulator operation to maximize initial flow rates to the interceptor and maintain underflow during the event. Final flush effects were visually noted on one occasion but not analytically documented. "First flush" conditions included rapid depth increase and high flow rates which would scour most pipe segments nearly simultaneously. Conditions under which final flush might occur are limited to less pronounced depth changes and flow rates. Scouring may occur at various times along the sewer making a peak concentration undeterminable.

BOD and phosphorous data collected at the regulator appear to follow similar trends as the suspended solids results. The correlation coefficient for for paired solids and BOD data is 0.91. The correlation coefficient for solids and total phosphorous data is 0.99. It can be concluded that most of the BOD and phosphorous is tied up in the solids matter. Soluble phosphorous fractions of total phosphorous loads ranged between 20 and 93 per cent resulting in a poor total to soluble phosphorous correlation coeficient of 0.75. A trend was noted toward higher fractions of soluble portions of total phosphorous increasing from an average of 36% for "start storm" samples to 59% for "mid storm" samples and to 84% for post storm samples. This trend inversely follows the declining solids concentration values and indicates that soluble phosphorous is not related to solids concentrations.

For the single storm event where overflow to the river occurred, the period of overflow was brief such that only one sample was obtained and only phosphorous data could be generated. Total phosphorous concentration for the single "mid-storm" overflow sample was 30% of the composited "mid storm" underflow sample phosphorous concentration. Overflow to the river (surface water of the backwatered flow) was thus much less contaminated than water sampled at the mid depth sampler intake for the dry/wet weather flow channel. These percentages reaffirm the phenomenon of solids settling in the backwatered flow and passing to the interceptor and confirm the design concept.

9.6 Monitoring of the Weiss Street Pump Station

Modifications to the Weiss Street Pump Station were limited to the construction of a weir wall within the pump wet well area designed to eliminate or delay pump "turn on" during storm events. The delay was caused by preventing the West Side Interceptor flow from spilling (at about springline depth) into the pump station wet well. Backwatering would result as greater head would be needed to push flow thorugh the hydraulically constrained river crossing. Spillover would finally occur but only after a higher depth of flow had been reached. The advantage of this delay is that at both the beginning and the end of storm periods, greater flows rates will exist in the West to East Crossover Interceptor due to increased West Side water elevation. In concept, the crossover modification would considerably modify and lessen overflows for the smaller intensity storms and have appreciably lesser impact for large major storms. Thus an increase in the crossover rate should reduce the overall pumped overflow at the Weiss Street Pump Station.

Post-BMP plan system hydraulics are complicated by the regulator modifications which were intended to increase flows to the West Side Interceptor leading to the Weiss Street Pump Station. Net results of the opposing effects of increased flow to the Weiss Street Pump Station and increased crossover flow rates could, depending on event intensity, either increase or decrease actual pumpage volumes at the Weiss Street Station compared to pre-BMP conditions. The reasons for this flow regime are as follows. The CSO Facility Plan concept was to reduce the number and magnitude of individual regulator overflows to a minimum (by either maximizing in-system storage and/or by modifying overflow sidewalls within the regulators to enhance interceptor flows) and allowing any excessive interceptor flows during major storm events to be handled by CSO control configurations located at the major pump overflow stations. Such a wet weather overflow treatment facility was programmed for the Weiss Street area. Overall, it was envisioned that as part of the BMP design, pumped overflows volumes at the Weiss Street Station requiring treatment would be smaller for minor storm events and be greater for major events as compared with pre-BMP conditions.

Data collected during the post implementation period included total operating times of the pumps using the permanent time clocks and stormwater quality based on samples collected via a temporary sampler installed by EDP. Total pumpage was calculated, neglecting minor pump capacity changes due to variable sump levels, using a value of 100 cfs per pump. Samples were composited and tested for suspended solids, BOD and total/soluble phosphorous.

Figure 9.8 illustrates pumped discharge volumes of CSO to the Saginaw River at the Weiss Street Station. This figure was earlier used in Chapter 6 to indicate the rainfall to pumpage relationship and has been updated to include 1984 data for periods representing pre and post-BMP modifications at the upstream regulators and at the Weiss Street Pump Station. All 1984 data falls within the range of previous data. On this basis

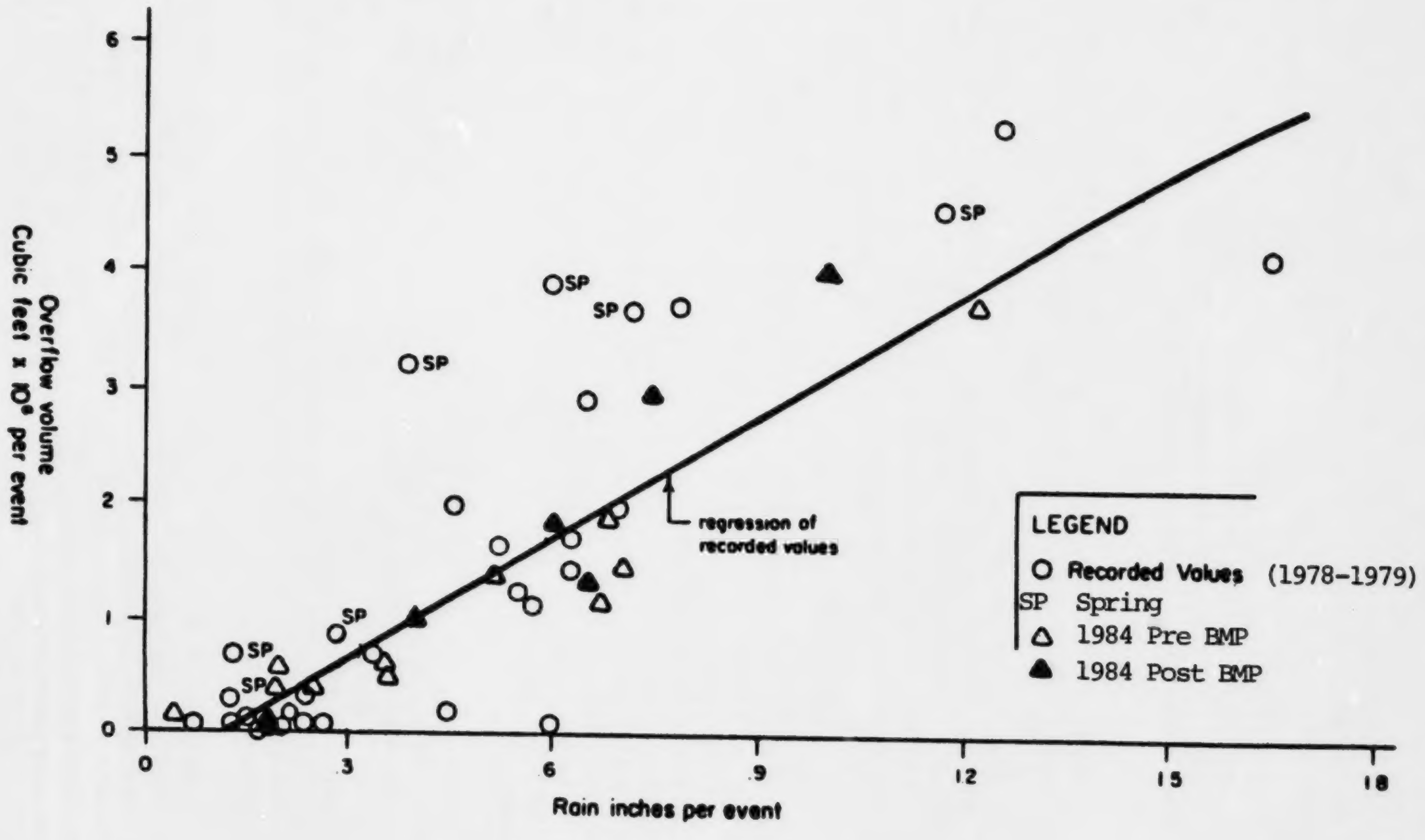


Figure 9.8 Overflow into Weiss Street pumping station

it would appear that the effects of increased flow to the interceptor at the regulator chambers has been substantially offset by the increased East/West crossover flow. There appears, however, some tendency for post BMP pumped flows to be less than pre BMP conditions for small storms and higher for rainfall events exceeding 0.7 inches.

As stated, the present data is limited and as such exact trends cannot be defined. In general, it can be concluded that for light to moderate intensity events Weiss Street CSO pumpage has not increased above pre modification rates. The important point is, that due to implementation of the partial BMP plan, overflows at numerous untreated locations were reduced or eliminated. Stormwater flow is thus directed at an increased rate to the WWTP where treatment is available with excess overflow concentrated at the Weiss Street Pump Station. The CSO Facility Plan concept requires that overflow treatment be provided in the future.

Water quality data at the Weiss Street Pump Station consisted of BOD, SS and total and soluble phosphorous analysis. Evaluation of phosphorous data was included in Chapter 6, where the primary purpose was to determine WWTP phosphorous levels and WWTP removal efficiencies. BOD and solids data were both field collected and predicted for each storm event using the simulation modeling program.

Figure 9.9 includes suspended solids concentration data for each event during the 1984 portion of the pre-implementation evaluation program. Data is seperated by period within an event including the "pre-peak" condition, during which time the flow rate in the system is increasing from base dry weather conditions to the peak flow rate, followed by the "mid-storm" condition when flow rates remain at elevated levels, and the "post" storm conditions (flow rates dropped to "pre-storm" conditions). Figure 9.9 clearly illustrates the effects of "first flush" conditions at the Weiss Street Station particularly for the first several events. High "pre-peak" solids concentrations are due to the combined effects of street washoff and scouring of system sediments. Lesser "mid storm" concentrations are due to dilutional effects of increased flow and a reduction in solids materials. "Post storm" levels are indicative of diluted sewage and minor amounts of remaining washoff materials.

The simulation model was used to estimate SS and BOD loadings at the Weiss Street Station for these monitored events. Average modeled BOD and SS overflow loadings equalled 86 and 75% of average measured loadings indicating reasonable closure.

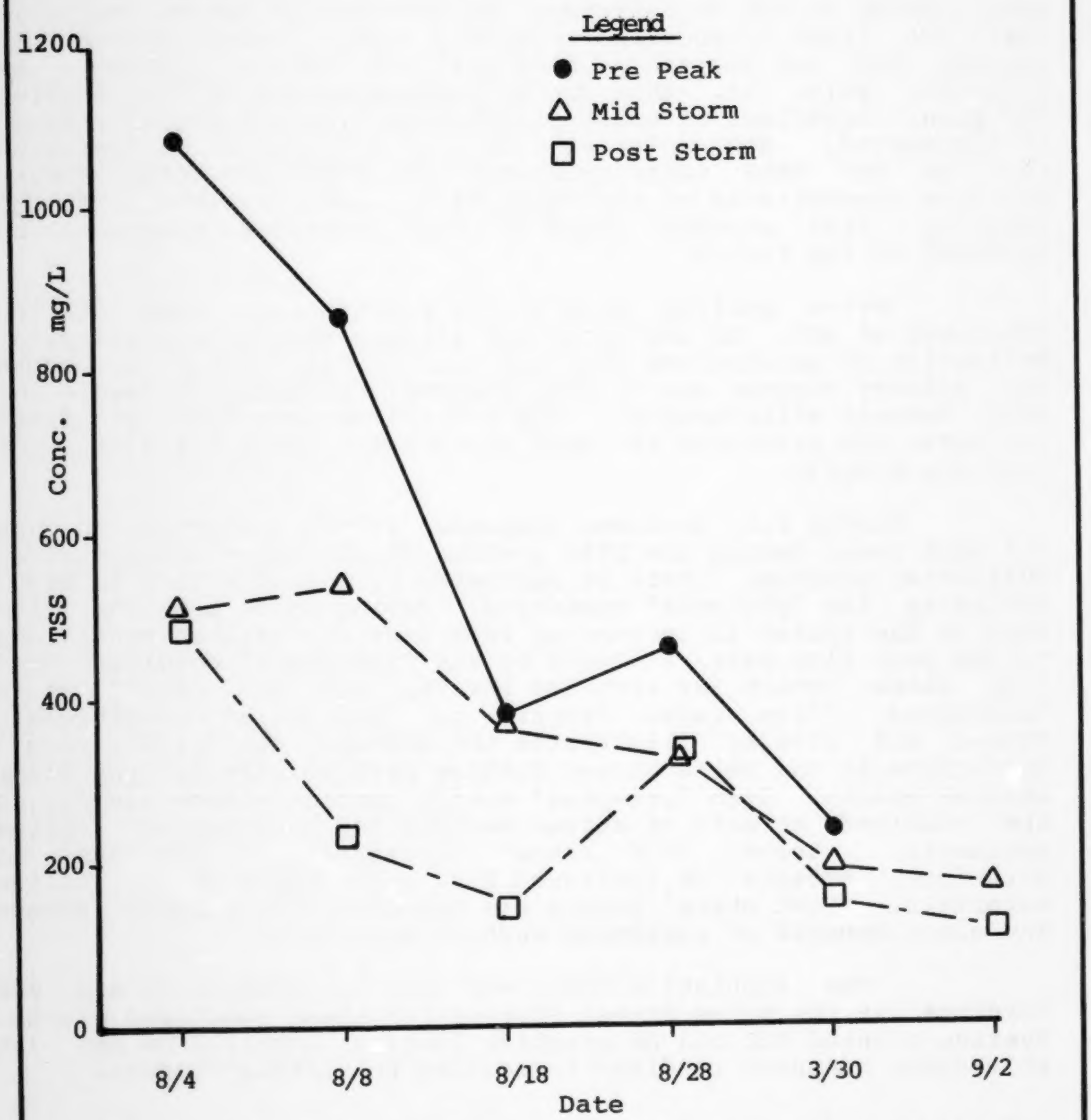


Figure 9.9 Weiss Street Pump Station Suspended Solids Data

9.7 Monitoring at the Waste Water Treatment Plant

Data collection at the WWTP consisted of continuous influent rate monitoring and total bypass flow during storm events and stormwater quality monitoring based on samples collected via a temporary sampler installation at the influent grit/overflow chamber. Samples were composited on a flow-weighed basis during the storm period.

The intent of much of the reduced BMP plan was to increase overall wet weather flows to the WWTP. Figure 9.10 illustrates cummulative WWTP wet weather flow volumes above normal dry weather flow for the day of an event, second and third calender days (cumulated) if flow rates had not returned to average dwf values. Discrete pre construction and post construction data points for the 1984 evaluation are separately depicted. Figure 9.10 indicates that most of both the pre and post construction data are similar although there are several events during the post construction period indicating a substantially increased flow at the WWTP. On this evidence it is difficult to definitely verify the effects of the BMP program elements implementation to date. It appears that there is a tendency for increased flows at the WWTP.

As with the Weiss Street data, the approach used was to ensure a reasonable correlation between the field results and predicted model data. With the model suitably calibrated, the effectiveness of the program implementation could be reviewed based on long term data simulations. Results of these simulations is presented in Chapter 10.

Comparison of WWTP field data to model data included the flow analysis discussed in Chapter 6, which resulted in minor modifications to the model and a good correlation of measured to modeled data. On a water quality basis the relative measured to model data are presented in Figure 9.11. Discrete data for pre construction and post construction periods are included. For both the suspended solids and BOD water quality parameters, the model reasonably predicted both pre/post implementation conditions.

9.8 Field Verification of Flow Regulating Device Calibration

Simulation of CSO backup and drain-down was performed at several of the modified chambers by EDP personnel to allow on-site calibration of the new vortex flow regulating devices. Physical chamber conditions precluded direct discharge measurements at most of the regulation chambers. Backup was produced either by closing the manual sluice gate or by placing a flat plate in front of the weir wall thimble. Depth measurements

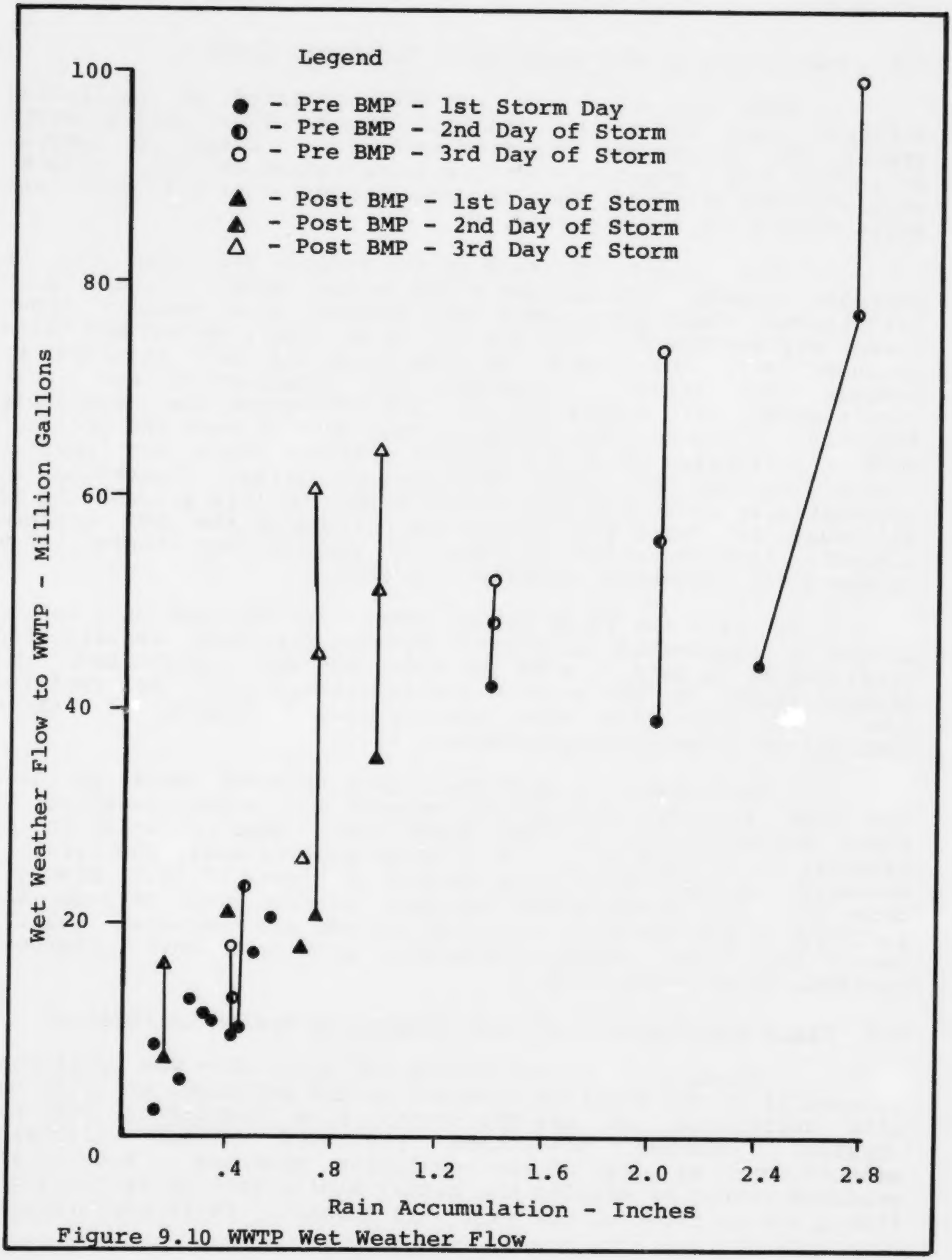


Figure 9.10 WWTP Wet Weather Flow

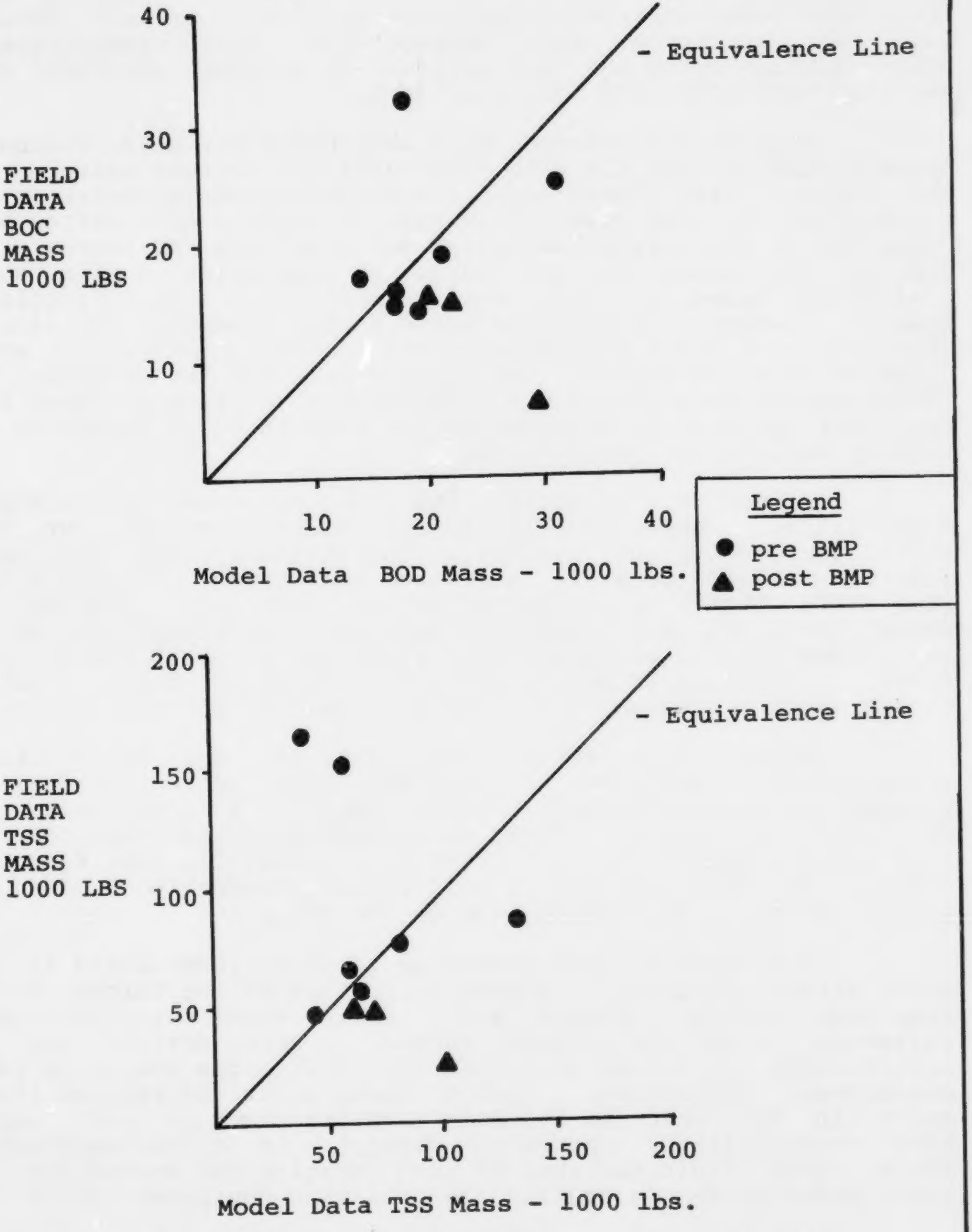


Figure 9.11 WWTP Wet Weather Water Quality Mass Data

were recorded at frequent intervals with simultaneous velocity measurements made with a Marsh-McBirney velocity probe at the flow regulator inlet pipe. Conversion of velocity data to flow rates was based on known inlet cross sectional areas. Results for each location were plotted on the manufacturer's head/discharge curve for each device. In general, the field data verified manufacturer's specifications.

Figures 9.12 through 9.16 illustrate the head discharge characteristics for the five flow regulator devices measured in the field. Each figure depicts both design characteristics and field data. In some cases field data is limited to a narrow head range due to the limited volume of dwf available for backwatering and in all cases by the range of operation during storm conditions based on side weir elevations. Each regulator chamber, except the modified Adams Street chamber, contains a sluiceway with a top elevation at the influent sewer invert and a depth of up to 30 inches. At Hayes Street and Throop Street the volume within this sluice was insufficient to allow low head flow regulator calibration considering the high rates of discharge and rapidly falling head conditions.

Figure 9.12 details the data for the Hayes Street installation. Data is limited to a minimum head of two feet which corresponds to the top of the sluiceway and the sewer invert. Field data is somewhat scattered but all within reasonable limits of the design curve. For this location the water level at the point of side weir spillover is at an equivalent head value of 3.5 ft. Thus, the range of field values shown in Figure 9.12 covers the entire range of unit operation during storm events and indicates good control of outflow rates.

Figure 9.13 details the data for the Adams Street installation. The range of field data shown include head values between the regulating device inlet crown and 9 inches below the spillover weir crest. Field data indicates that the observed flow pattern follows the characteristic curve. In sum, flow rate control at this location is well within reasonable limits for proper operation of this portion of the BMP plan.

The vortex valve discharge rates field-measured at the Cronk Street regulator , Figure 9.14, and at the Throop Street regulator device, Figure 9.15, also demonstrate reasonable agreement with the design curves. Data scatter may be attributable to actual unit accuracy limitations and/or to field measurement techniques. Each of these locations was monitored twice in the field and two data sets are shown on each curve. Data repeatability, though not perfect, is quite reasonable. Field notes indicated that at each location the second run was less prone to debris fouling the measuring equipment since the

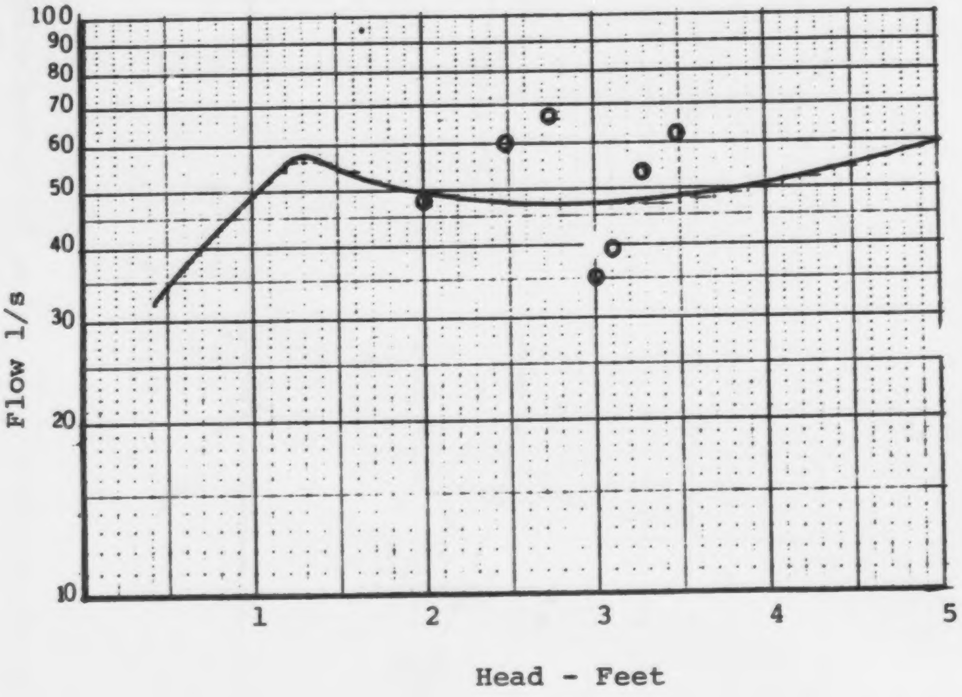


Figure 9.12 Hayes Street Vortex Valve Characteristics

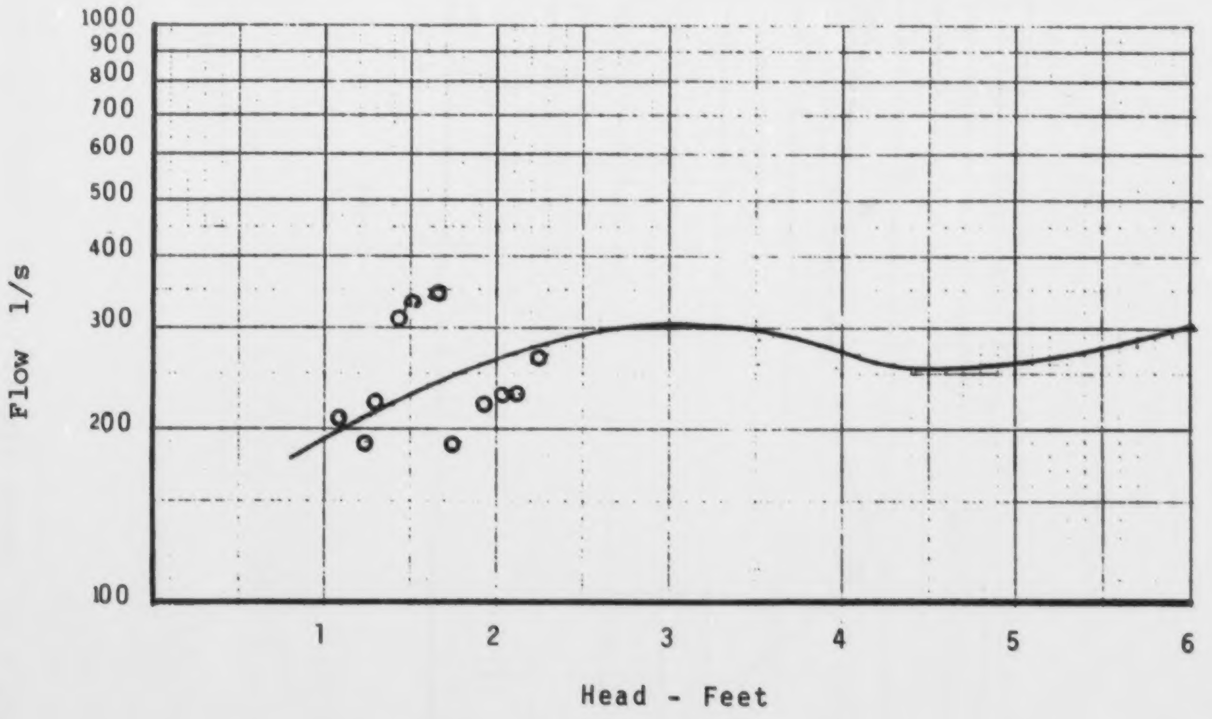


Figure 9.13 Adams Street Vortex Valve Characteristics

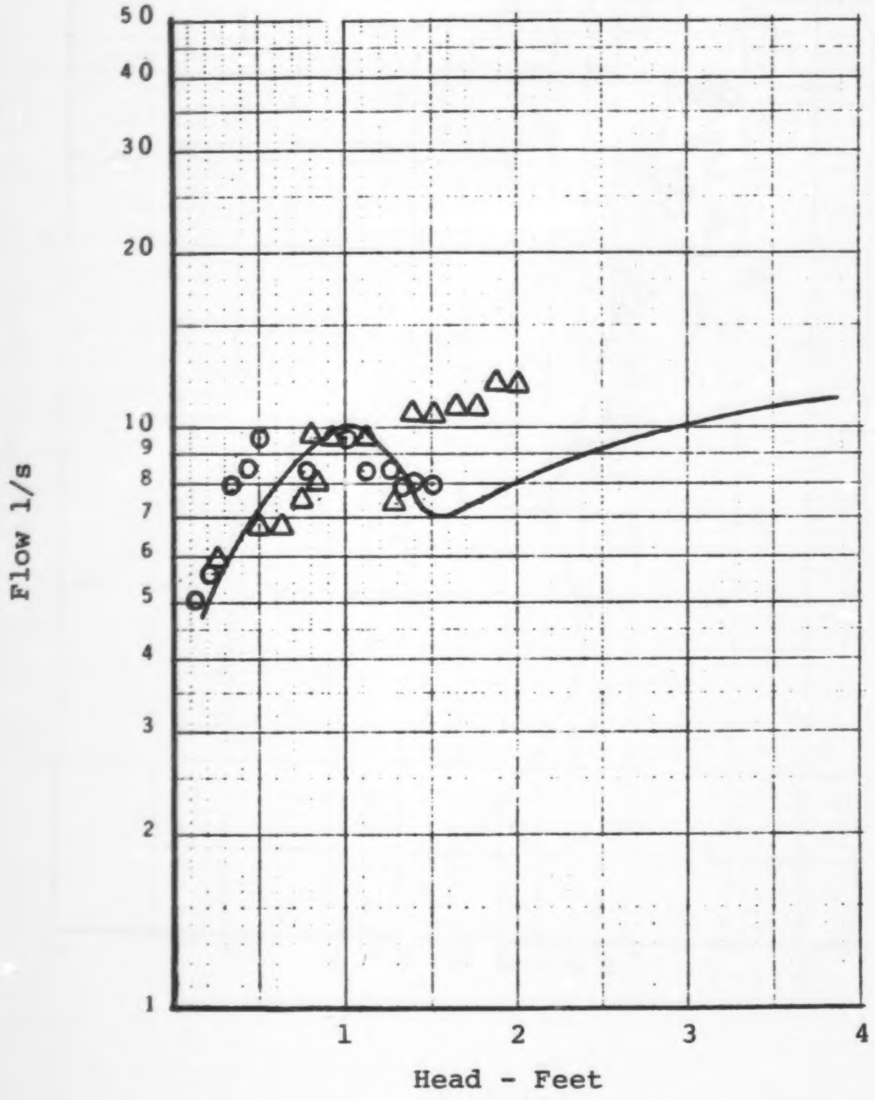


Figure 9.14 Court Street Vortex Valve Characteristics

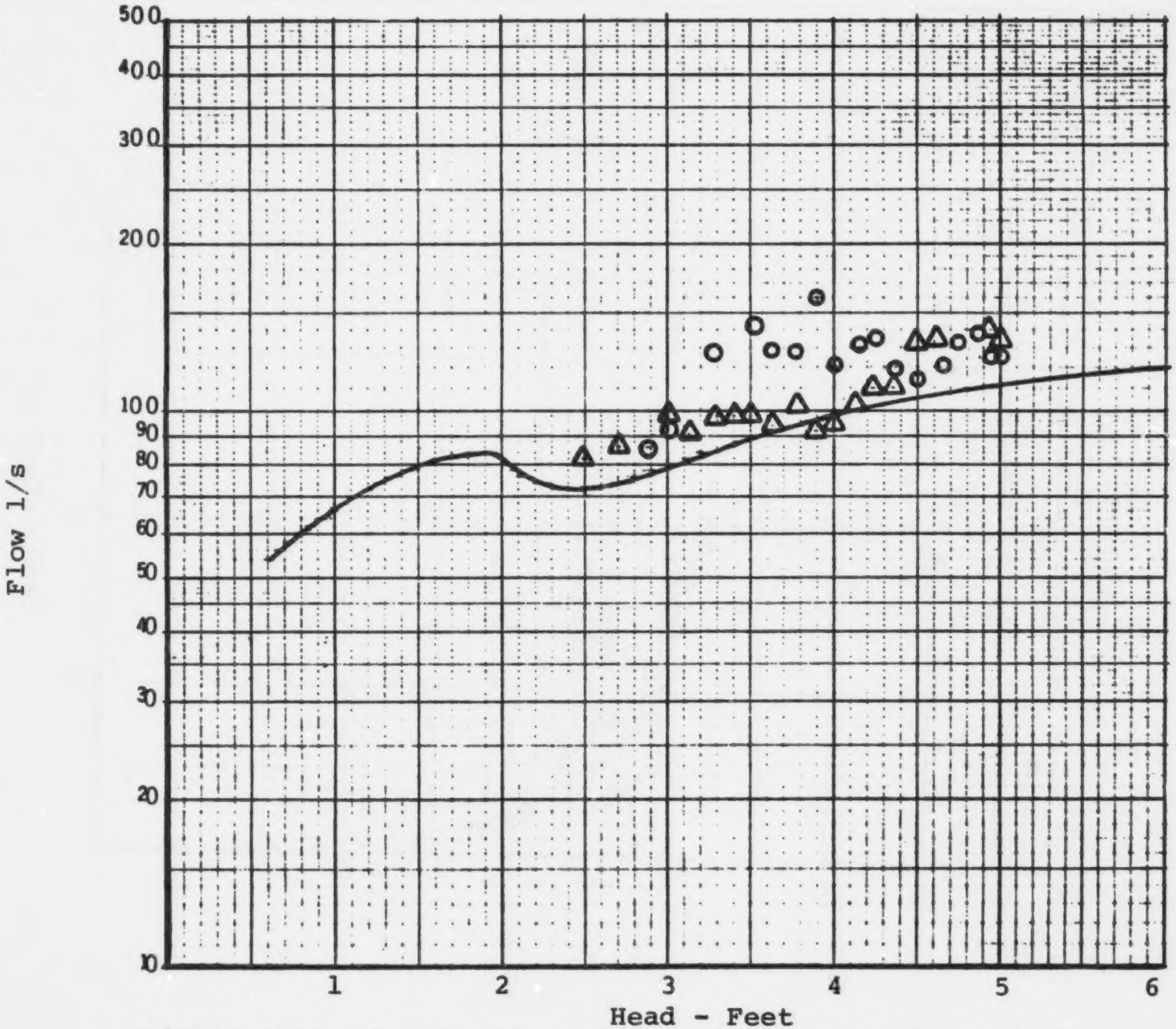


Figure 9.15 Throop Street Vortex Valve Characteristics

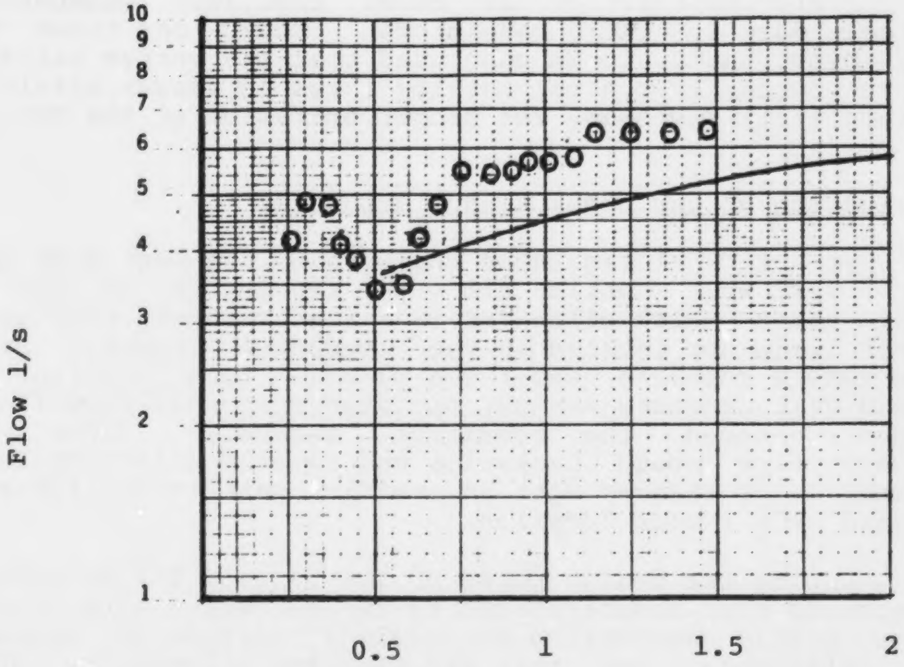


Figure 9.16 Ames Street Vortex Valve Characteristics

first run promoted scouring of the sewer line and allowed for relatively clean water flow during the second run.

The four flow regulator units investigated above were the conical configuration type devices. The device installed at Ames Street is of a different design commonly termed the "horizontal" type as the vortex flow pattern is developed in a horizontal plane. Field data shown in Figure 9.15 fall slightly above the vortex flow regulator design head/discharge curve. The significant aspect of the field data is that it reasonably matches design data shown and demonstrates a narrow range of flow rate for the head conditions encountered in this application.

Field calibration at other regulator chambers was impractical due to field conditions. Based on those units actually investigated, it is concluded that the vortex valve flow regulators adequately control outflow rates to ranges within the design conditions necessary for proper operation of the BMP plan.

9.9 Evaluation of the Cronk Avenue Regulator

Two original regulator chambers on the West Side of the Saginaw River were unlike all other regulators in that the backwater gates which allow for excess stormwater overflow to the river were not located in the regulator chamber, but in separate remote chambers nearer the river. Each location, Cronk Avenue and West Genessee Avenue, operated such that flow from two directions entered the regulator chamber. Under storm conditions, flow would backwater and reverse direction in one feed line. A chamber with an overflow gate to the river was constructed at a remote location.

During the design phase of the reduced BMP program, it was determined that modifications to the backwater gate chambers would be costly considering the minimal increase in stormwater storage attainable, and that all flow not allowed to gravity spill to the river might be pumped as overflow at the Weiss Street Pump Station at additional cost. These modifications were not deemed desireable until satellite wet weather treatment is provided at the Weiss Street Pump Station. Modification at these chambers was limited to replacement of the existing float-operated regulating devices with new vortex valve throttling devices.

Post reduced BMP plan implementation evaluation included monitoring of the depth of flow at the Cronk Avenue backwater gate chamber and the peak depth at a location upstream in the Cronk Avenue drainage area. Six events were monitored during the period each demonstrating similar characteristics of stormwater draindown rates. Draindown periods consistently lasted several days such that in some cases drainage was not complete at the start of a subsequent event. Flow to the river appears to have occurred for all events which ranged in total accumulation from 0.17 to 0.77 inches and in duration from 2 hours to 12 hours. The limited storage available is indicated by the apparent spillover to the river after 4 hours for an event with 0.17 inches accumulated over 5 hours.

The draindown periods were assumed attributable to the upstream hydraulic/hydrologic conditions. Based on the estimated storage volume of 49,000 cu.ft., the estimated dry weather flow rate and the vortex valve design discharge rate, a draindown period of 12 hours is predicted. The 65 to 75 hour periods field recorded appear to be attributable to either continued inflow/infiltration to this part of the Saginaw sewerage system, a higher than estimated dry weather flow base rate or both.

Two upstream off-line storage chambers are located within the drainage area. The Congress Avenue chamber consists of approximately 1300 ft. of 77 in. by 121 in. pipe and provides about 86,000 cu. ft. of storage. The second chamber is actually located in the Township area (above the City of Saginaw limits) and consists of an underground concrete structure of rectangular shape providing about 100,000 cu. ft. of storage. During extreme events creating surcharge conditions, combined sewage overflows side weirs of the main 60 inch trunk line into the Congress Avenue chamber. City personnel release the stored water after events by manually opening a butterfly valve. City records indicate that for the period of October and November, 1984 no overflow to the chamber occurred. At the Jameson Street chamber pumping of stored water is required. Although pump records were available it was calculated that the volume of storage in the Jameson Street chamber or even the combined storage volumes of both chambers were insufficient to account for the volume of flow noted after each event at the Cronk regulator.

Based on a detailed evaluation of available data, it appears the long draindown periods are due to a current dry weather flow rate including infiltration, much greater than previously estimated during Phase 1 and Phase 2. The only adverse effects of the existing conditions are the lack of storage volume available for secondary storm events. No adverse problems have been reported in terms of excessive surcharge or basement flooding under modified conditions. In addition, it is intended that both the Cronk Avenue and West Genessee Avenue chambers be modified concurrently with the implementation of treatment at the Weiss Street Station.

9.10 Post Construction Chamber Inspection and Evaluation

Evaluation of new flow regulating devices and chamber modifications was performed during the post construction period. Post storm observations were by City of Saginaw maintenance personnel.

Observations were recorded at each modified regulator chamber after each storm event. Records indicate the relative intensity of the storm event, flow conditions in the chamber upstream and downstream of the new regulating device and conditions at the device. Observations were made of the sewage flow to determine if free flow conditions existed without blockage. All chamber areas and related piping were inspected for debris accumulation in areas subject to storm conditions and for debris sedimentation in areas of dry weather flow.

Operating conditions at most chambers were reported as good with little or no debris accumulation or sedimentation. One general exception is the occurrence of debris remaining on the top surface of concrete weir walls. During the design phase this possibility was noted, however as these surfaces also act as walkways within the chamber it was deemed preferable to allow small amounts of debris accumulation than to produce unsafe footing by sloping or crowning weir tops. Debris accumulations were predicted and have been found to be insufficient to cause odor problems.

Locations with adverse conditions reported on one or more occasions included the Ames Street regulator chamber, the Court Street regulator chamber and the Salt/Vermont Streets in-line regulator chamber. At the Ames Street regulator chamber near complete blockage of the wall orifice leading to the flow regulator was reported on one occasion and blockage of the device on two other occasions. Apparently large deposits of scale or grease were present and the City personnel cleaned the area and the device. It is believed these conditions resulted from a combination of factors, including large amounts of grease within a relatively small discharge waste stream. The particular flow through pattern of the horizontal type vortex valve may have had an additional negative impact since no cloggage problems have been reported for the conical type device at the Court Street chamber which passes similar low flow rates (0.33 cfs) but not large quantities of grease noted at Ames Street. Although it is difficult to conclude from these observations that the horizontal type device should not be used in the future for very low flow rates having high grease content, care and discretion should be used when this type of device is used to regulate small sewerage flows.

Construction of the new chamber at the Salt/Vermont Street location included the use of concrete fill material to contour all chamber areas in an effort to prevent debris and sediment accumulation. After construction several areas were noted to be improperly sloped and debris accumulation has resulted in the production of odors in the chamber. Corrective action requires only additional concrete material to match design conditions and should eliminate problem areas. Otherwise the entire chamber and new regulator have been reported to operate quite satisfactorily. No upstream deposition problems have been noted as a result of the in-line storage generated at this location.

The Cronk Street regulator chamber is one location where persistent problems related to the new regulating device have been reported. On several occasions the chamber has been backwatered due to flow blockage resulting from sticks, rags and other materials lodged within the flow regulating device and at the sluice gate. The problem is compounded by the difficult access at this location. Weir walls extent to within 17 inches of the chamber ceiling making human access difficult and no access manhole originally existed for the area between the weir walls. The City has since provided a new access manhole to the chamber above the sluiceway simplifying the unclogging process, when necessary.

Blockages at the Cronk Street regulator appear to be due to an unusual amount of large floating materials from the upstream drainage area. The high weir walls within the chamber prevent any overflow to the interceptor. Accumulations of large floatable material may ensue and during final draindown combine to clog the regulating device.

During the design phase of the project it was determined that altering the remote backwatering gate and the chamber side weirs at the Cronk Street Chamber would not be cost effective at this location considering the limited storage available and the need to pump excess flow at the Weiss Street Station. The intent was to make these modifications after treatment was provided at Weiss Street. The problem of large floatable accumulations due to the lack of an overflow area was not anticipated. It is recommended that the situation be observed further and resolved, if found necessary, either by lowering the weir walls to allow overflow of large floatable debris or by installing an upstream stick-trapping device.

In general, the new regulating devices have demonstrated the ability at most of the installations to pass typical sewage and CSO materials. Any items physically too large to pass through the device may become trapped.

CHAPTER 10

ASSESSMENT OF PRE BMP CONDITIONS AND EVALUATION OF PROPOSED PROGRAMS

10.1 Foreword

The present water quality of the Saginaw River and Bay is determined by a wide variety of events both induced by man and of natural origin. CSO are only one aspect of the general problem. However, CSO have been shown to be significant in their deleterious impacts yet controllable. Although the proposed activities will improve river water quality and the degree of improvement can be assessed for selected parameters, general water quality improvements will be of a wider nature than can be presently quantified.

10.2 Overview of Analysis Criteria and Results Presentation

Assessment of "pre BMP" conditions makes use of EDP's stormwater runoff simulation model to estimate pollutant emission to Saginaw River. The simulation runs present annual summaries of combined sewer overflows and loadings at each overflow point, and for the system in general. Overflows and loadings are considered to be those resulting from wet weather rainfall events. Snowmelt and infiltration during non-rainfall periods are implicitly included in the dry weather flow and are not tallied in the wet weather loadings.

Dry weather loadings at the WWTP are considered adequately handled by the plant and not part of the CSO problem. Total wet weather rainfall impacted loadings to the river include both loadings caused by overflows in the system upstream of the WWTP, as well as the WWTP loadings during rainfall events (excluding normal dry weather effluent contributions). The 1978 WWTP removal efficiencies for TSS and BOD of 93% and 89%, respectively, were considered maintainable during storm events and were used to calculate the residual stormwater loadings from the WWTP. Review of wet weather data for the September to November, 1984 post construction period indicated TSS removals to average 93.3% and BOD to average 86.1%. TSS removals were therefore consistent with earlier 1978 data. Lower BOD values during wet weather were apparently consistent with slightly lower overall efficiencies. The three month period averaged 87.4% BOD removal efficiency. On this basis it was assumed that no change in removal efficiency for TSS or BOD resulted from the reduced plan implementation and the original yearly average removal rates were attainable and as such, used for model simulations. As previously discussed, phosphorous loadings to the WWTP were based on a nominal influent concentration of 1.3 mg/l and WWTP removal efficiencies of 50 and 75 percent for treatment without and with pickling liquor treatment. Phosphorous loadings at overflow points were based on a 1.74 mg/l concentration.

Table 10.1 presents the "pre BMP" model run and offers an example of a typical simulation run summary. Part "A" of the summary table will typically present overflow and related discharge information while part "B" will present overflow pollutant (TSS and BOD) loadings to the Saginaw River. Results are always presented for the entire period simulation (for 1977, there are 78 rainfall events), for "low" rainfall events defined as less than 0.5 inches of accumulated rainfall (for 1977, there are 60 such events), and for "high" rainfall events defined as greater than 0.5 inches of accumulated rainfall (for 1977, there are 18 such events).

For each of the three periods overflow and water quality loadings are presented for seven major overflow points, for the rest of the overflow points on the East and West Sides, and for flows and loadings influent to the WWTP. Gravity and pumped discharges are lumped together for the Fourteenth Street overflow. The Weiss Street gravity overflow discharge is separately reported from pumped overflows at the Weiss Street Pumping Station. All quantities reported from overflow locations are effluent discharges to the Saginaw River after treatment, if applicable. For example, overflow and pollutant loadings from the Hancock Street Station reflect excess overflow discharge from the sedimentation units not bledback to the WWTP. Peak WWTP flows reported such as in Table 10.1 represent the highest hourly average flow rate into the WWTP. Reported mean and standard deviations are computed on the basis of non-zero events. Event durations ranged between one hour and three days for the 78 storms in 1977. Yearly discharge volumes and pollutant loadings are also reported. Total hours reported in the tables represent the total number of hours that some overflow occurred (gravity or pumped). All flow volumes are in thousands of cubic feet. All pollutant loadings are in thousands of pounds. Average river elevations were used for all runs unless otherwise noted (580.15 feet - USGS datum).

10.3 Pre BMP CSO Emission Estimates

Simulation analysis results for "pre BMP" conditions, that is, the original system configuration with Hancock Street Facility are given in Table 10.1. Although the "pre BMP" simulation was also run during Phase 1 and Phase 2 of the overall project (1978-1983), results presented are those based on the

TABLE 10.1A

PRE BMP CONDITIONS SIMULATION

FLOWS - OVERFLOWS

PERIOD : 1 / 6 TO 12 / 26 78 EVENTS 33 INCHES TOTAL RAIN

WWTP	FULL PERIOD	LOW RAIN ***	HIGH RAIN ***
PEAK FLOW	128	128	128
VOLUME *	158968	84398	74570
MEAN **	2038	1407	4143
STND DEV	1707	741	2254
NO.EVENTS	78	60	18

OVERFLOW POINTS

	FULL PERIOD VOLUME*	LOW RAIN *** VOLUME*	HIGH RAIN *** VOLUME*
FRASER	76 EVENTS 405 HRS	58 EVENTS 230 HRS	18 EVENTS 175 HRS
YEAR TOT	11199	2507	8692
MEAN **	147	43	483
STND DEV	391	58	698
HANCOCK	16 EVENTS 50 HRS	5 EVENTS 9 HRS	11 EVENTS 41 HRS
YEAR TOT	15131	1930	13200
MEAN **	946	386	1200
STND DEV	1390	204	1607
WEISS	76 EVENTS 405 HRS	58 EVENTS 230 HRS	18 EVENTS 175 HRS
YEAR TOT	64306	23010	41296
MEAN **	846	397	2294
STND DEV	1525	269	2616
WEISS PUMP	63 EVENTS 358 HRS	45 EVENTS 161 HRS	18 EVENTS 197 HRS
YEAR TOT	59013	19375	39368
MEAN **	937	431	2202
STND DEV	1126	394	1344
WEBBER	20 EVENTS 49 HRS	5 EVENTS 7 HRS	15 EVENTS 42 HRS
YEAR TOT	5110	317	4793
MEAN **	255	63	320
STND DEV	557	43	630
EMERSON	0 EVENTS 0 HRS	0 EVENTS 0 HRS	0 EVENTS 0 HRS
YEAR TOT	0	0	0
MEAN **	0	0	0
STND DEV	0	0	0
FOURTEENTH	77 EVENTS 504 HRS	59 EVENTS 280 HRS	18 EVENTS 224 HRS
YEAR TOT	10129	2507	7622
MEAN **	132	42	423
STND DEV	306	41	532
WEST RESID	76 EVENTS 354 HRS	58 EVENTS 198 HRS	18 EVENTS 156 HRS
YEAR TOT	41249	8319	32930
MEAN **	543	143	1829
STND DEV	2031	94	3901
EAST RESID	76 EVENTS 405 HRS	58 EVENTS 230 HRS	18 EVENTS 175 HRS
YEAR TOT	55659	16684	38974
MEAN **	732	288	2165
STND DEV	1685	225	3023

* - 1000 CUBIC FEET - ABOVE DRY WEATHER FLOW ** - PER NON-ZERO EVENT
*** - LOW RAIN 0.5 INCHES OR LESS - HIGH RAIN GREATER THAN 0.5 INCHES

TABLE 10.1B

PRE BMP CONDITIONS SIMULATION

WATER QUALITY

PERIOD : 1 / 6 TO 12 / 26 78 EVENTS 33 INCHES TOTAL RAIN

WWTP	FULL PERIOD		LOW RAIN ***		HIGH RAIN ***	
	TSS *	BOD *	TSS *	BOD *	TSS *	BOD *
TOT LOAD	3075	939	1673	605	1402	334
MEAN **	39.4	12.0	27.9	10.1	77.9	18.5
STND DEV	31.7	7.3	15.2	5.0	40.8	9.4

OVERFLOW POINTS	FULL PERIOD		LOW RAIN ***		HIGH RAIN ***	
	TSS *	BOD *	TSS *	BOD *	TSS *	BOD *
FRASER						
TOT LOAD	254	35	60	13	194	22
MEAN **	3.3	0.5	1.0	0.2	10.8	1.2
STND DEV	8.8	0.8	1.3	0.2	15.8	1.3
HANCOCK						
TOT LOAD	305	36	31	6	274	30
MEAN **	19.1	2.2	6.2	1.1	24.9	2.8
STND DEV	30.8	3.0	5.1	0.7	35.5	3.5
WEISS						
TOT LOAD	1590	496	646	258	944	238
MEAN **	20.9	6.5	11.1	4.5	52.4	13.2
STND DEV	34.2	6.7	6.5	2.3	59.3	10.7
WEISS PUMP						
TOT LOAD	1001	317	308	126	693	191
MEAN *MEAN **	15.9	5.0	6.8	2.8	38.5	10.6
STND DEV	20.4	5.0	7.1	2.4	24.8	5.4
WEBBER						
TOT LOAD	116	17	7	2	109	16
MEAN **	5.8	0.9	1.4	0.3	7.2	1.0
STND DEV	12.8	1.6	0.9	0.2	14.5	1.8
EMERSON						
TOT LOAD	0	0	0	0	0	0
MEAN **	0.0	0.0	0.0	0.0	0.0	0.0
STND DEV	0.0	0.0	0.0	0.0	0.0	0.0
FOURTEENTH						
TOT LOAD	221	63	57	22	163	41
MEAN **	2.9	0.8	1.0	0.4	9.1	2.3
STND DEV	6.7	1.3	0.9	0.3	11.8	2.2
WEST RESIDUAL						
TOT LOAD	891	137	198	56	692	81
MEAN **	11.7	1.8	3.4	1.0	38.5	4.5
STND DEV	43.1	4.1	1.9	0.4	83.1	7.8
EAST RESIDUAL						
TOT LOAD	1178	245	375	115	803	130
MEAN **	15.5	3.2	6.5	2.0	44.6	7.2
STND DEV	35.6	4.4	4.5	1.0	64.6	7.5

* - 1000 POUNDS ** - PER NON-ZERO EVENT

*** - LOW RAIN 0.5 INCHES OR LESS - HIGH RAIN GREATER THAN 0.5 INCHES

final model editions as calibrated in Phase 3 (1984). Regulators were set to operate as they originally existed. In-line storage was composed of the storage generated by the five modified regulators in the Hancock Street area, and the available storage (up to the point backwater gates would open) for all other regulators. The interceptor system operated as originally perceived with the Weiss Street Station wet well weir level unadjusted. The WWTP flow rate was throttled to allow only a throughput maximum flow of 128 cfs, the maximum rate observed during the 1984 field investigation.

The total annual wet weather rainfall impacted combined sewage discharge for 1977 delivered to the WWTP and to all the overflow/pumping station locations equals 420.8 million cubic feet. Approximately 37.7% of this total or 159.0 million cubic feet is delivered to the WWTP with an untreated balance of 261.8 million cubic feet. The fraction of the combined sewage discharge at the WWTP during 60 low rainfall events is 53.0% of the overall low rainfall total (140.1 million cubic feet) and is 28.5% for the 18 high rainfall events. The high rainfall event subtotal equals 62.1% of the annual overall combined sewer overflows.

The "pre BMP" overflow summary results in Table 10.2 indicate that the Weiss Street area (Weiss Street Pump Station and the Weiss Street gravity overflow from the Township) is the primary overflow in the system representing 47.1% of the annual system CSO (excluding discharge to the WWTP). The Hancock Street overflow is second, representing 5.8% of the total untreated CSO suggesting that the bottleneck at the river crossing has a substantial impact at both the Hancock and Weiss Street Station discharge points. Overflows at Webber Street, Fourteenth Street and Fraser Street represent 2.0%, 3.9% and 4.3%, respectively, of the annual CSO discharge. The Emerson Street Station was not used during any events. The remainder of the system's CSO discharge points from both the East and West Side regulators accounted for 37.1% of the total, 21.3% and 15.8% respectively.

The "pre BMP" total annual (1977) wet weather TSS and BOD pollutant loadings generated in the system and delivered to the WWTP and all overflow/pumping station locations equal 8.63 and 2.29 million pounds, respectively (See Tables 10.3 and 10.4). TSS and BOD loadings to the WWTP represent 35.6% and 41.1% of the overall wet weather load. (Tables 10.3 & 10.4) After WWTP treatment, however, the residual wet weather WWTP TSS and BOD loadings represent less than 10% of the total wet weather loadings to the river. Pollutant emissions from Hancock and Weiss Street locations are the dominant source of organic loadings to the river from the major overflow points. Pollutant emissions from the 60 low rainfall events represent roughly 54%

TABLE 10.2

FLOW SUMMARY DATA

	PRE BMP TOTAL FLOW		BMP TOTAL FLOW		REDUCED BMP TOTAL FLOW	
	1000 FT^3	*%	1000 FT^3	%	1000 FT^3	%
WEISS	123,319	47.1	95,861	66.0	81,404	37.1
HANCOCK	15,131	5.8	27,228	18.7	30,017	13.7
FRASER	11,199	4.3	5,558	3.8	5,376	2.4
14TH	10,129	3.9	12,855	8.9	13,296	6.1
WEBBER	5,110	2.0	3,752	2.6	4,144	1.9
EMERSON	0	0	0	0	0	0
EAST	55,659	21.3	0	0	55,709	25.4
WEST	41,249	15.8	0	0	29,036	13.4
SUBTOTAL	261,796	100	145,254	100	218,982	100
WWTP	158,968		303,823		240,508	
TOTAL	420,764		449,077		459,490	

SUMMARY STATISTICS

	PRE BMP	BMP	REDUCED BMP
WWTP % OF TOTAL FLOW	37.7	67.7	52.3
WEISS % OF TOTAL FLOW	29.3	21.3	17.7

* % OF SUBTOTAL FLOW - EXCLUDING WWTP

TABLE 10.3

TSS SUMMARY DATA

LOCATION	PRE BMP TOTAL LOAD 1000 LBS	BMP TOTAL LOAD 1000 LBS	REDUCED BMP TOTAL LOAD 1000 LBS
WWTP LOAD	3,075	6,068	5,140
WWTP REMOVAL	2,860	5,643	4,780
WWTP NET TO RIVER	215	325	360
WEISS TO RIVER	2,591	2,053	1,778
ALL OTHER TO RIVER	2,965	1,031	2,892
TOTAL WET WEATHER LOAD	8,631	9,152	9,810
TOTAL LOAD TO RIVER	5,771	3,509	5,030

SUMMARY STATISTICS

WWTP % OF TOTAL WET WEATHER LOAD	35.6	66.3	52.4
WWTP % REMOVAL OF WWTP LOAD	93	93	93
WWTP % REMOVAL OF TOTAL LOAD	33.1	61.7	48.7
WWTP RESIDUAL % OF RIVER LOAD	3.7	9.3	7.2
CONTROL PROGRAM NET % REMOVAL	0	28.6	15.6

TABLE 10.4

BOD SUMMARY DATA

LOCATION	PRE BMP TOTAL LOAD 1000 LBS	BMP TOTAL LOAD 1000 LBS	REDUCED BM TOTAL LOAD 1000 LB
WWTP LOAD	939	1,944	1,802
WWTP REMOVAL	836	1,730	1,604
WWTP NET TO RIVER	103	214	198
WEISS TO RIVER	813	492	466
ALL OTHER TO RIVER	533	163	547
TOTAL WET WEATHER LOAD	2,285	2,599	2,815
TOTAL LOAD TO RIVER	1,449	869	1,211

SUMMARY STATISTICS

WWTP % OF TOTAL WET WEATHER LOAD	41.1	74.8	64.0
WWTP % REMOVAL OF WWTP LOAD	89	89	89
WWTP % REMOVAL OF TOTAL LOAD	36.6	66.6	57.0
WWTP RESIDUAL % OF RIVER LOAD	7.1	24.6	16.4
CONTROL PROGRAM NET % REMOVAL	0	30.0	20.4

of the total implying that considerable pollutant removal gain can be achieved by control of overflow during low level storms. The annual "pre BMP" wet weather related TSS and BOD loadings to the Saginaw River (for the simulation year of 1977) equal 5.77 and 1.45 million pounds, respectively. See Tables 10.3 and 10.4.

10.4 Analysis of Weiss Weir Alteration

"Pre BMP" conditions were also run during Phase 2 simulations (1983) with the Weiss Street regulator weir raised 5 feet to assess its impact on the system. Summary results of the earlier simulations are reported here to indicate the significance of the weir modification.

Model operation simulating the Weiss Street Pump Station weir alteration as the only change to the Saginaw system indicated that volumes of overflow at specific locations would change, particularly at the WWTP where total flow increased 20.2%. The flow increase to the WWTP is mainly offset by the substantial change in pumpage requirements at Weiss Street. Increasing the Weiss Street Station weir elevation also acts to pond more water in the West Side Interceptor system as evidenced by a minor increase in overflow at the Hancock Street Station. The flow increase at Hancock Street exactly balanced the WWTP increase and the Weiss Street decrease. No changes were recorded in the simulation results for any of the other large overflow points or for totals of the smaller regulator chambers on either river side.

Considering the effectiveness of the WWTP in handling wet weather flow, and using the Phase 3 "pre BMP" model volumes, overall reductions in wet weather loadings potentially gained from the single act of raising the Weiss Street weir height are 0.58 and 0.17 million pounds for TSS and BOD, which are 10.0% and 9.2% of the present total loadings.

The results support the hypothesis that the interceptor river crossing is a major bottleneck to flow. Restricted pumpage rates at the WWTP wet well quickly back up the East Side Interceptor which, in turn throttles flow across the river and backs up the West Side Interceptor. An important step to relieve the system is to reduce overflow at the Weiss Street Pump Station by increasing the weir height into the wet well, and thereby increase the flow across the river and to the WWTP. Excess system capacity appears available in the Emerson area, suggesting diversion of all the smaller East Side overflow points to the East Side interceptor with pumping relief and overflow treatment provided at the Emerson Street Pumping Station. This action should also increase the flow to the WWTP.

10.5 Complete BMP Program Simulation

The complete BMP program incorporates all the low level structural improvements and management practices that could be implemented at relatively low cost. The concept of the program is to maximize the use of existing facilities before further CSO treatment facilities are implemented. The program maximizes in-line storage already available in the system conduits and maximizes the conveyance of storm flows to the WWTP. To effectuate this plan, the 33 existing regulators are modified, 14 new flow control devices are installed within new control chambers to increase in-line storage, and weir modifications are accomplished at the Weiss Street Pump Station to increase flow across the river.

Complete BMP implementation includes modification of the Type "B" regulators to yield a constant discharge during initial storage filling, with excess flows beyond available storage diverted to the interceptor rather than the river. Modifying the Type "B" regulator chambers includes adding a cross weir between the backwater gate and the flow channel, and altering the existing side weirs between the flow channel and the drain system leading to the interceptor. In this manner flows in excess of what is discharged through the new vortex valve flow regulating device or stored in the system behind the weir, will first overflow the lower weir and be directed to the interceptor. Only extremely high flows will overflow the cross weir separating the backwater gate portion of the chamber, and be directed to the river. This weir arrangement was considered by EDP to be both the least expensive and most available method of limiting overflows at all of the regulator chambers in the system. The modified Hancock area regulators are also to have the automatic sluice gates leading to the river replaced with flap gates. Six additional regulators of smaller size (Type "A") are to be altered with weirs to divert all flows to the interceptor. Alterations to produce this effect were proposed under the previous Hancock area modification plan in 1977 and it is the intent of the BMP program to complete these modifications.

The regulator chamber at Fraser Street is to be unaltered except for removal of the float operated swing gate regulator. An increased weir elevation of five feet is to be imposed at the Weiss Street Pumping Station to reduce pumped overflows at that point and increase the flow across the river. Fourteen upstream structures are to be constructed and vortex valve regulating devices are to be installed to yield additional in-line flow control for storage upstream of the existing regulator chambers. The resulting changes to the regulator operations achieve the maximum in-line storage potential of the system without causing system flooding.

The only points of overflow as a result of these actions are at Fraser, Hancock, Weiss (pump and gravity), Webber, Emerson and Fourteenth Streets except in extreme flooding conditions. Weir elevations at Webber and Fourteenth Street overflow points would be raised slightly to insure that the fairly high post rain discharges from upstream in-line storage devices are totally diverted to the interceptor. An upstream flow control device is intended only a short distance upstream.

The total annual wet weather rainfall impacted combined sewage discharge for 1977 delivered to the WWTP and to all the overflow/pumping station locations equals 449.0 million cubic feet as simulated by the Phase 3 (1984) model for the complete BMP program (see Table 10.5). Approximately 67.7% of this total or 303.8 million cubic feet is delivered to the WWTP with an untreated balance of 145.2 million cubic feet. The fraction of the combined sewage discharge to the WWTP during 60 low rainfall events is 87.5% of the overall low rainfall total and is 50.3% for the 18 high rainfall events. The high rainfall event subtotal equals 53% of the annual overall combined sewer overflows.

The complete BMP plan overflow summary results in Table 10.2 indicate that the Weiss Street area represents 66% of the annual system CSO (excluding discharge to the WWTP). The Hancock Street overflow represents 18.7% of the total. Overflows at Webber Street, Fourteenth Street and Fraser Street represent 2.6%, 8.9% and 3.8%, respectively, of the annual CSO discharge. The complete BMP plan would eliminate discharge of CSO from all other points on both the East and West Sides.

The complete BMP plan total annual (1977) wet weather TSS and BOD pollutant loadings generated in the system and delivered to the WWTP and all overflow/pumping station locations equal 9.15 and 2.6 million pounds, respectively. See Tables 10.3 and 10.4. TSS and BOD loadings at the WWTP represent 66.3% and 74.8% of the overall wet weather total before treatment. After WWTP treatment, the residual wet weather WWTP TSS and BOD loadings represent only 9.3% and 24.6% of the total wet weather loadings to the river. Overall, the WWTP reduces the TSS and BOD loadings by 61.7 and 66.6%, respectively, resulting in wet weather related discharges of 3.5 and 0.87 million pounds of TSS and BOD, respectively, to the Saginaw River. Pollutant emissions from Hancock and Weiss Street locations are the remaining dominant source of organic loadings to the river from the major overflow points.

Comparing the above flow and pollutant summary data to that developed earlier for the "pre BMP" simulation indicates substantial reductions in overall CSO loadings to the river

TABLE 10.5A

COMPLETE BMP PLAN SIMULATION

```
                              FLOWS - OVERFLOWS
PERIOD :  1 / 6  TO  12 / 26     78  EVENTS   33  INCHES TOTAL RAIN
WWTP              FULL PERIOD            LOW RAIN ***        HIGH RAIN ***
 PEAK FLOW                 128                    128                  128
 VOLUME *               303823                 183031               120792
 MEAN **                  3895                   3051                 6711
 STND DEV                 2633                   1782                 3020
 NO.EVENTS         78                    60                   18
OVERFLOW POINTS
                   FULL PERIOD          LOW RAIN ***        HIGH RAIN ***
                        VOLUME*               VOLUME*              VOLUME*
FRASER       18 EVENTS 73 HRS    4 EVENTS 7 HRS       14 EVENTS 66 HRS
 YEAR TOT                 5558                    203                 5355
 MEAN **                   309                     51                  383
 STND DEV                  650                     39                  719
HANCOCK      9 EVENTS 37 HRS     0 EVENTS 0 HRS       9 EVENTS 37 HRS
 YEAR TOT                27228                      0                27228
 MEAN **                  3025                      0                 3025
 STND DEV                 5207                      0                 5207
WEISS        46 EVENTS 237 HRS   28 EVENTS 94 HRS     18 EVENTS 143 HRS
 YEAR TOT                40646                   8929                31718
 MEAN **                   884                    319                 1762
 STND DEV                 1686                    210                 2435
WEISS PUMP 63 EVENTS 362 HRS     45 EVENTS 1 63 EVENTS 362 HRS
                                 45 EVENTS 168 HRS    18 EVENTS 194 HRS
 YEAR TOT                55215                  12719                42496
 MEAN **                   876                    283                 2361
 STND DEV                 1529                    355                 2186
WEBBER       11 EVENTS 39 HRS    1 EVENTS 1 HRS       10 EVENTS 38 HRS
 YEAR TOT                 3752                     34                 3718
 MEAN **                   341                     34                  372
 STND DEV                  661                      0                  686
EMERSON      0 EVENTS 0 HRS      0 EVENTS 0 HRS       0 EVENTS 0 HRS
 YEAR TOT                    0                      0                    0
 MEAN **                     0                      0                    0
 STND DEV                    0                      0                    0
FOURTEENTH 76 EVENTS 837 HRS     58 EVENTS 492 HRS    18 EVENTS 345 HRS
 YEAR TOT                12855                   4073                 8782
 MEAN **                   169                     70                  488
 STND DEV                  349                     56                  609
WEST RESID 0 EVENTS 0 HRS        0 EVENTS 0 HRS       0 EVENTS 0 HRS
 YEAR TOT                    0                      0                    0
 MEAN **                     0                      0                    0
 STND DEV                    0                      0                    0
EAST RESID 34 EVENTS 127 HRS     16 EVENTS 36 HRS     18 EVENTS 91 HRS
 YEAR TOT                    0                      0                    0
         0                   0                      0
 MEAN **                     0                      0                    0
 STND DEV                    0                      0                    0
* - 1000 CUBIC FEET - ABOVE DRY WEATHER FLOW  ** - PER NON-ZERO EVENT
*** - LOW RAIN 0.5 INCHES OR LESS - HIGH RAIN GREATER THAN 0.5 INCHES
```

TABLE 10.5B

COMPLETE BMP PLAN SIMULATION

WATER QUALITY

PERIOD : 1 / 6 TO 12 / 26 78 EVENTS 33 INCHES TOTAL RAIN

WWTP	FULL PERIOD		LOW RAIN ***		HIGH RAIN ***	
	TSS *	BOD *	TSS *	BOD *	TSS *	BOD *
TOT LOAD	6068	1944	3728	1364	2340	580
MEAN **	77.8	24.9	62.1	22.7	130.0	32.2
STND DEV	47.2	13.5	31.0	12.0	54.0	15.7

OVERFLOW POINTS	FULL PERIOD		LOW RAIN ***		HIGH RAIN ***	
	TSS *	BOD *	TSS *	BOD *	TSS *	BOD *
FRASER						
TOT LOAD	125	12	4	1	120	12
MEAN **	6.9	0.7	1.1	0.2	8.6	0.8
STND DEV	14.8	1.2	0.8	0.1	16.4	1.3
HANCOCK						
TOT LOAD	559	53	0	0	559	53
MEAN **	62.1	5.9	0.0	0.0	62.1	5.9
STND DEV	108.9	10.3	0.0	0.0	108.9	10.3
WEISS						
TOT LOAD	944	254	225	81	719	173
MEAN **	20.5	5.5	8.0	2.9	39.9	9.6
STND DEV	38.1	7.2	4.7	1.5	55.4	10.1
WEISS PUMP						
TOT LOAD	1109	238	252	77	857	161
MEAN **	17.6	3.8	5.6	1.7	47.6	8.9
STND DEV	31.5	4.7	7.2	2.0	45.7	5.4
WEBBER						
TOT LOAD	85	12	1	0	84	12
MEAN **	7.7	1.1	0.7	0.1	8.4	1.2
STND DEV	15.2	1.9	0.0	0.0	15.8	2.0
EMERSON						
TOT LOAD	0	0	0	0	0	0
MEAN **	0.0	0.0	0.0	0.0	0.0	0.0
STND DEV	0.0	0.0	0.0	0.0	0.0	0.0
FOURTEENTH						
TOT LOAD	268	86	91	37	177	49
MEAN **	3.5	1.1	1.6	0.6	9.8	2.7
STND DEV	7.3	1.6	1.2	0.4	12.9	2.6
WEST RESIDUAL						
TOT LOAD	0	0	0	0	0	0
MEAN **	0.0	0.0	0.0	0.0	0.0	0.0
STND DEV	0.0	0.0	0.0	0.0	0.0	0.0
EAST RESIDUAL						
TOT LOAD	0	0	0	0	0	0
MEAN **	0.0	0.0	0.0	0.0	0.0	0.0
STND DEV	0.0	0.0	0.0	0.0	0.0	0.0

* - 1000 POUNDS ** - PER NON-ZERO EVENT

*** - LOW RAIN 0.5 INCHES OR LESS - HIGH RAIN GREATER THAN 0.5 INCHES

(Tables 10.3, 10.4). Simulations of the 1977 storm events indicate that implementing the total BMP would reduce the annual "Pre BMP" wet weather induced TSS loading to the Saginaw River by 28.6%, the BOD by 30%. As detailed below, implementation of a reduced BMP plan results in slightly lower overall reductions due to increased overflows at the smaller regulators.

10.6 Reduced BMP Program

The reduced BMP program incorporated most of the low level structural improvements of the complete BMP program for the West Side. No alterations are recommended on the East Side as the numerous simulations performed demonstrated that implementation of East Side improvements in full or in part, will result in excess flow to the Emerson Street facility. Under "pre BMP" flow conditions, use of this manually operated facility was virtually unnecessary and one preference of the reduced BMP program was to avoid increased manpower requirements. West Side regulators were modified as discussed under the complete BMP program with weirs set to elevations as detailed in Chapter 7. The five foot weir elevation increase at the Weiss Street Pump Station was completed.

One of the original fourteen upstream in-line storage locations was selected as a demonstration site for the construction of a new flow regulation/backwater/storage facility. This location is upstream of the existing Fraser Street regulator and provides an estimated 175,000 cubic feet of storage, a major reason for selection of this site.

Utilizing the reduced BMP program including the Weiss Street Pump Station weir adjustment, the total annual wet weather rainfall impacted combined sewage discharge for 1977 delivered to the WWTP and to all the overflow/pumping station locations equals 459.5 million cubic feet (see Tables 10.6 and 10.2). Approximately 52.3% of this total or 240.5 million cubic feet is delivered to the WWTP with an untreated balance of 219 million cubic feet. The fraction of the combined sewage discharge to the WWTP during 60 low rainfall events is 72.1% of the overall low rainfall total and is 39% for the 18 high rainfall events. The high rainfall event subtotal equals 59.1% of the annual overall combined sewer overflows.

The flow summary results in Table 10.4 indicate that the Weiss Street area described earlier as the primary overflow in the system represents 37.1% of the annual system CSO (excluding discharge to the WWTP). The Hancock Street overflow represents 13.7% of the total. Overflows at Webber Street, Fourteenth Street and Fraser Street represent 1.9%, 6.4% and

TABLE 10.6A

REDUCED BMP PLAN SIMULATION

```
-----------------------------------------------------------------------------
                             FLOWS - OVERFLOWS

PERIOD :  1 / 6  TO  12 / 26     78  EVENTS   33  INCHES TOTAL RAIN

-----------------------------------------------------------------------------
WWTP               FULL PERIOD            LOW RAIN ***         HIGH RAIN ***
 PEAK FLOW                   128                   128                   128
 VOLUME *                 240508                135226                105282
 MEAN **                    3083                  2254                  5849
 STND DEV                   2336                  1387                  2690
 NO.EVENTS          78                    60                    18
-----------------------------------------------------------------------------
OVERFLOW POINTS
                   FULL PERIOD           LOW RAIN ***          HIGH RAIN ***
                          VOLUME*               VOLUME*             VOLUME*
FRASER      18 EVENTS 71 HRS      4 EVENTS 6 HRS        14 EVENTS 65 HRS
 YEAR TOT                   5376                   160                  5216
 MEAN **                     299                    40                   373
 STND DEV                    652                    39                   722
-----------------------------------------------------------------------------
HANCOCK     14 EVENTS 56 HRS      3 EVENTS 9 HRS        11 EVENTS 47 HRS
 YEAR TOT                  30017                  1186                 28831
 MEAN **                    2144                   395                  2621
 STND DEV                   4351                   107                  4799
-----------------------------------------------------------------------------
WEISS       76 EVENTS 587 HRS     58 EVENTS 348 HRS     18 EVENTS 239 HRS
 YEAR TOT                  43037                 10533                 32504
 MEAN **                     566                   182                  1806
 STND DEV                   1402                   230                  2473
-----------------------------------------------------------------------------
WEISS PUMP  48 EVENTS 257 HRS     30 EVENTS 95 HRS      18 EVENTS 162 HRS
 YEAR TOT                  38367                 10238                 28129
 MEAN **                     799                   341                  1563
 STND DEV                    866                   306                   955
-----------------------------------------------------------------------------
WEBBER      13 EVENTS 49 HRS      2 EVENTS 3 HRS        11 EVENTS 46 HRS
 YEAR TOT                   4144                    99                  4045
 MEAN **                     319                    50                   368
 STND DEV                    630                    11                   673
-----------------------------------------------------------------------------
EMERSON     0 EVENTS 0 HRS        0 EVENTS 0 HRS        0 EVENTS 0 HRS
 YEAR TOT                      0                     0                     0
 MEAN **                       0                     0                     0
 STND DEV                      0                     0                     0
-----------------------------------------------------------------------------
FOURTEENTH  76 EVENTS 791 HRS     58 EVENTS 486 HRS     18 EVENTS 305 HRS
 YEAR TOT                  13296                  4109                  9187
 MEAN **                     175                    71                   510
 STND DEV                    359                    61                   621
-----------------------------------------------------------------------------
WEST RESID  76 EVENTS 692 HRS     58 EVENTS 418 HRS     18 EVENTS 274 HRS
 YEAR TOT                  29036                  9265                 19771
 MEAN **                     382                   160                  1098
 STND DEV                    762                   125                  1316
-----------------------------------------------------------------------------
EAST RESID  76 EVENTS 483 HRS     58 EVENTS 279 HRS     18 EVENTS 204 HRS
 YEAR TOT                  55709                 16759                 38951
 MEAN **                     733                   289                  2164
 STND DEV                   1688                   234                  3029
-----------------------------------------------------------------------------
 * - 1000 CUBIC FEET - ABOVE DRY WEATHER FLOW  ** - PER NON-ZERO EVENT
 *** - LOW RAIN 0.5 INCHES OR LESS - HIGH RAIN GREATER THAN 0.5 INCHES
```

TABLE 10.6B

REDUCED BMP PLAN SIMULATION

```
------------------------------------------------------------------------------
                                  WATER QUALITY

PERIOD :  1 / 6  TO  12 / 26     78 EVENTS     33 INCHES TOTAL RAIN
------------------------------------------------------------------------------
WWTP               FULL PERIOD           LOW RAIN ***          HIGH RAIN ***
                 TSS *       BOD *     TSS *       BOD *     TSS *       BOD *
 TOT LOAD         5140        1802      3041        1233      2100         568
 MEAN **          65.9        23.1      50.7        20.6     116.7        31.6
 STND DEV         44.1        12.7      27.2        10.9      51.2        14.4
------------------------------------------------------------------------------
OVERFLOW POINTS
                   FULL PERIOD           LOW RAIN ***          HIGH RAIN ***
                 TSS *       BOD *     TSS *       BOD *     TSS *       BOD *
FRASER
 TOT LOAD          121          12         3           0       117          11
 MEAN **           6.7         0.7       0.9         0.1       8.4         0.8
 STND DEV         14.8         1.2       0.8         0.1      16.4         1.3
------------------------------------------------------------------------------
HANCOCK
 TOT LOAD          612          61        17           2       595          59
 MEAN **          43.7         4.4       5.7         0.8      54.1         5.3
 STND DEV         92.0         8.7       2.7         0.3     101.4         9.6
------------------------------------------------------------------------------
WEISS
 TOT LOAD         1008         277       271         100       737         177
 MEAN **          13.3         3.6       4.7         1.7      41.0         9.9
 STND DEV         31.8         6.3       5.4         1.8      56.2        10.2
------------------------------------------------------------------------------
WEISS PUMP
 TOT LOAD          770         189       212          66       558         123
 MEAN **          16.1         3.9       7.1         2.2      31.0         6.8
 STND DEV         17.4         3.2       6.3         1.9      19.5         2.8
------------------------------------------------------------------------------
WEBBER
 TOT LOAD           94          13         2           0        92          13
 MEAN **           7.2         1.0       1.1         0.2       8.3         1.2
 STND DEV         14.5         1.8       0.2         0.0      15.5         2.0
------------------------------------------------------------------------------
EMERSON
 TOT LOAD            0           0         0           0         0           0
 MEAN **           0.0         0.0       0.0         0.0       0.0         0.0
 STND DEV          0.0         0.0       0.0         0.0       0.0         0.0
------------------------------------------------------------------------------
FOURTEENTH
 TOT LOAD          291          87        94          37       197          50
 MEAN **           3.8         1.1       1.6         0.6      10.9         2.8
 STND DEV          7.9         1.6       1.3         0.4      13.8         2.5
------------------------------------------------------------------------------
WEST RESIDUAL
 TOT LOAD          613         133       206          65       407          68
 MEAN **           8.1         1.7       3.5         1.1      22.6         3.8
 STND DEV         16.0         1.9       2.5         0.6      27.9         2.9
------------------------------------------------------------------------------
EAST RESIDUAL
 TOT LOAD         1161         241       367         112       795         129
 MEAN **          15.3         3.2       6.3         1.9      44.1         7.2
 STND DEV         35.5         4.4       4.6         1.0      64.4         7.5
------------------------------------------------------------------------------
   * - 1000 POUNDS          ** - PER NON-ZERO EVENT
  *** - LOW RAIN 0.5 INCHES OR LESS - HIGH RAIN GREATER THAN 0.5 INCHES
```

2.4%, respectively, of the annual CSO discharge. The Emerson Street Station was not used during any events. The remainder of the system's CSO discharge points on the East Side accounted for 25.4% of the total. CSO discharged from the West Side regulators accounted for 13.4%. These CSO are due to limited structural changes at the Cronk Street and W. Genessee Avenue regulators. Presently, available storage at these locations is limited and excess flow if not allowed to pass by gravity to the river would require pumping at the Weiss Station. Until the control facility is implemented at Weiss Street to handle pumped overflows, it was not deemed appropriate to alter the present overflow patterns.

The reduced BMP program annual (1977) wet weather TSS and BOD pollutant loadings generated in the system and delivered to the WWTP and all overflow/pumping station locations equal 9.81 and 2.82 million pounds, respectively (see Tables 10.3 and 10.4). TSS and BOD loadings at the WWTP represent 52.4% and 64.0% of the overall wet weather total before WWTP. After WWTP treatment, the residual wet weather WWTP TSS and BOD loadings represent only 7.2% and 16.4% of the total wet weather loadings to the river. Overall, the WWTP reduces the loading of TSS and BOD by 48.7 and 57%, respectively resulting in wet weather related discharges of 5.03 and 1.21 million pounds of TSS and BOD, respectively, to the Saginaw River. Pollutant emissions from Hancock and Weiss Street locations are the remaining dominant source of organic loadings to the river from the major overflow points.

Comparing the above flow and pollutant summary data to that developed earlier for the "pre BMP" simulation indicates substantial reductions in overall CSO loadings to the river. Simulations of the 1977 storm events indicate that implementing the reduced BMP plan lowered the annual "Pre BMP" wet weather induced TSS loadings to the Saginaw River by 15.6% and the BOD loadings by 20.4%. Comparison of these values to those detailed at the end of Sections 10.4 and 10.5 indicate removals to be greater than that attained by altering the Weiss Street regulator chamber weir and less than that provided by the complete BMP program.

10.7 Phosphorous Analysis

Assessment of total phosphorous loadings and potential reductions under various simulation schemes is summarized in Table 10.7. A nominal WWTP concentration of 1.3 mg/l was set for stormwater flows to the WWTP under "pre BMP " conditions. Removal efficiencies recorded at the WWTP were typically 50% for secondary treatment and 75% with AWT phosphorous treatment. These rates were used in the computations. The total phosphorous

TABLE 10.7
TOTAL PHOSPHOROUS SUMMARY DATA

LOCATION	PRE BMP lbs	REDUCED BMP lbs	COMPLETE BMP lbs
WWTP LOAD*	12,900	21,351	28,331
WWTP AVG CONC MG/L	1.3	1.42	1.44
WWTP BYPASS (2%)	258	427	566
ADD FROM HANCOCK	213	423	384
WWTP REMOVAL 50%	6,248	10,674	14,074
(75%)	(9,641	(16,010)	(21,112)
WWTP NET TO RIVER 50%	6,685	10,676	14,650
(75%)	(3,472)	(5,341)	(7,603)
HANCOCK LOAD	1,642	3,257	2,955
HANCOCK REMOVAL (13%)	213	423	384
HANCOCK NET TO RIVER	1,429	2,833	2,571
WEISS TO RIVER	13,386	8,836	10,405
ALL OTHER TO RIVER	13,389	11,675	2,406
TOTAL WET WEATHER LOAD**	41,317	45,119	44,097
TOTAL LOAD TO RIVER 50% WWTP REMOVAL	34,889	34,020	30,032
(75% WWTP REMOVAL)	(31,676)	(28,685)	(22,985)

SUMMARY STATISTICS

WWTP % OF TOTAL LOAD	31.2	47.3	61.8
WWTP % REMOVAL RATE	50/75	50/75	50/75
WWTP % REMOVAL OF TOTAL LOAD 50% RATE	15.5	23.6	31.8
(75% RATE)	(23.3)	(35.5)	(47.9)
HANCOCK % REMOVAL OF TOTAL LOAD	0.5	0.9	0.9
NET % REMOVAL OF TOTAL LOAD 50% RATE	16.0	24.5	32.7
(75% RATE)	(23.8)	(36.4)	(48.8)
CONTROL PROGRAM NET % REMOVAL 50% RATE	0	8.5	16.7
(75% RATE)	(0)	(12.6)	(25.0)

* WWTP LOAD BASED ON 1.3 mg/l, ALL OTHER AT 1.74 mg/l

** TOTAL LOAD IS PRE BMP TOTAL LOAD FACTORED BY MODEL VOLUME RATIOS. WWTP IS TOTAL LOAD MINUS OTHER LOADS AT 1.74 mg/l concentration at all untreated overflow points was set at 1.73 mg/l.

Phosphorous loadings to the WWTP for other than "pre BMP" conditions were determined as the "pre BMP" total system load less the estimated loading to the river from the untreated overflows. Average concentrations at the WWTP were then calculated based on total WWTP flow and total phosphorous loading. On this basis the total phosphorous concentration in the WWTP influent was estimated as 1.44 mg/l for the complete BMP and 1.42 mg/l for the reduced BMP programs, respectively. Table 10.7 presents summary results of estimated total phosphorous loadings and removals.

Inspection of Table 10.7 indicates that implementation of the complete BMP program is estimated to increase the relative wet weather phosphorous loadings to the WWTP by 30.6% up to 61.8% of the total area load. At this rate, 32.7% removal would be expected, 16.7% above existing conditions, assuming 50% removal at the WWTP. Under conditions providing 75% phosphorous removal (teritary treatment - pickling liquor), the total phosphorous removal percentage attained would be 48.8% or 25% above existing conditions.

Performing the construction and alteration measures outlined in the reduced BMP program will provide 50% of the phosphorous reduction gain estimated attainable by the complete BMP program. The WWTP would receive 47.3% of the wet weather system phosphorous load and remove 24.5% of the total load at the 50% rate and 36.4% of the total load at the 75% rate. Incremental removal gains above pre BMP conditions would be 8.5% and 12.6%, respectively.

Based on an annual total wet weather load of 41,000 lbs. the total estimated phosphorous loadings to the river would be 34,000 lbs. for the reduced BMP plan and 30,000 lbs. for the complete BMP plan with secondary (50%) treatment of the WWTP. Using pickling liquor (or FeSO4) for AWT treatment, the residuals would be about 29,000 lbs. and 23,000 lbs., for the reduced BMP and complete BMP plans, respectively. Figure 10.1 graphically illustrates the relative annual wet weather phosphorous loadings to the Saginaw River based on various treatment senarios. Two sets of bar graphs were used to indicate estimated removals with the WWTP providing secondary (top of figure) and AWT treatment (bottom of figure). In addition, the soluble fraction of the total phosphorous is illustrated. The total wet weather load of 41,000 pounds per year is shown as 100%. River loadings for pre BMP, reduced BMP, and proposed complete BMP conditions are then shown in terms of pounds and percentages of soluble and total phosphorous. The upper figure represents removal rates based on standard secondary treatment. Phosphorous removal is due to the non soluble fraction tied in with solids materials. The lower figure illustrates the effects of AWT treatment for phosphorous.

Soluble fractions are reduced as shown but no additional non-soluble reduction occurs. Although some phosphorous remvoal is provicded with solids settling at Hancock Hancock, most of the reduction occurs at the WWTP. Figure 2.1 graphically illustrates substantial reductions in total phosphorous loadings to the river due to treatment at the WWTP and that implementation of BMP measures greatly enhance total removals. Soluble phosphorous removal is non-existent with the WWTP providing only secondary treatment.

AWT treatment at the WWTP using pickling liquor is shown in Figure 2.1 as it further reduces total phosphorous loadings to the river. The entire decrease is due to reduction of the soluble phosphorous fraction.

CHAPTER 11

APPLICATION OF INLET CONTROL TECHNOLOGY TO THE CITY OF SAGINAW COMBINED SEWERAGE SYSTEM

11.1 Foreword

Surcharging sewer/basement flooding problems, which can occur during storm conditions within sewerage systems of all types (seperated, adjacent and combined), have been identified in many parts of the country. The extent of these problems has led to investigations in the area of inlet control or stormwater management to develop inexpensive solutions in comparison to traditional outlet sewer relief sewer programs.

The focus of the Saginaw sewerage system evaluation so far has been on the incidence and reduction of CSO to the Saginaw River. For this reason most of the control technologies have been focused near the river with only partial emphasis on upstream control. Upstream control concepts have been limited to recommendations of in-line structures intended to create in-line storage.

Basement flooding from surcharging collection systems has been reported from time to time in the City and this chapter represents a brief review of this problem and considers potentially suitable inlet-control technologies and their effects on mitigating basement flooding and reducing CSO.

Section 11.2 describes the philosophy of inlet control technology. Section 11.3 details the methods used in the evaluation and further details the concepts of control technologies. Stormwater induced problems reported upstream in the City of Saginaw sewerage system are reviewed in Section 11.4. The Congress Courts area was selected for detailed analysis as described in Section 11.5. City-wide implementation of inlet control technology is considered in Section 11.6.

11.2 Inlet Control Philosophy

In the simplest of terms "inlet control" starts at the very top of a sewershed and controls/manages sewage/stormwater so that the pipes do not overload as you move downstream. All possible forms of above/underground storage are used to hold the water till the receiving pipe can handle the flow without surcharge. If sufficient potential cannot be feasibly generated then additional conveyance capacity, i.e., relief sewers are necessary. The analysis begins at the top of the system and progressively moves downstream to the outlet. If the sewer outlet is backwatered, then the conveyance capacity has to be adjusted to properly reflect outlet deficiencies.

The aim of the stormwater runoff control program is to develop surface and subsurface controls such that the sewerage system would discharge under full pipe conditions (or acceptable surcharge levels) during any given design storm without causing any adverse minor street surface ponding and/or adverse overland flow across private property. Basement flooding protection for a given design storm becomes equatable with ensuring that the rehabilitated sewer system during a given design storm would flow at acceptable levels with no adverse street ponding/overland flow.

The following set of control strategies are typically considered in stormwater management (inlet control) investigations:

a) Manhole rehabilitation,
b) Placement of overland flow training berms and depressed gutters,
c) Placement of surface storage berms to maximize surface storage potential,
d) Reconstruction of low curbs to maximize surface storage potential,
e) Placement of vortex valves into catchbasins to either maximize street surface storage or to create controlled overland flow that can be intercepted by downstream catchbasins and/or underground storage,
f) Construction of new catchbasins to intercept overland flow,
g) Construction of shallow drains to dewater low "trapped" street areas,
h) Construction of underground off-line storage tanks to detain excess surface runoff and/or spilled overflow from sewerage system,
i) Disconnection of residential downspouts,
j) Placement of Eave-N-Flow eavestrough regulators to restrict downspout flows, and
k) Placement of orifice restricting segments or weirs or vortex valves into sewerage system as to create controlled surcharge conditions.

Figure 11.1 pictorially shows some of "inlet control" methods. The various methods and/or the implements, that is, devices used in the implementation of "inlet control", can be broadly classified as "small scale technology alternatives for flooding relief". For example, a widely employed method of "inlet control" is to reduce direct uncontrolled flow to the sewer by

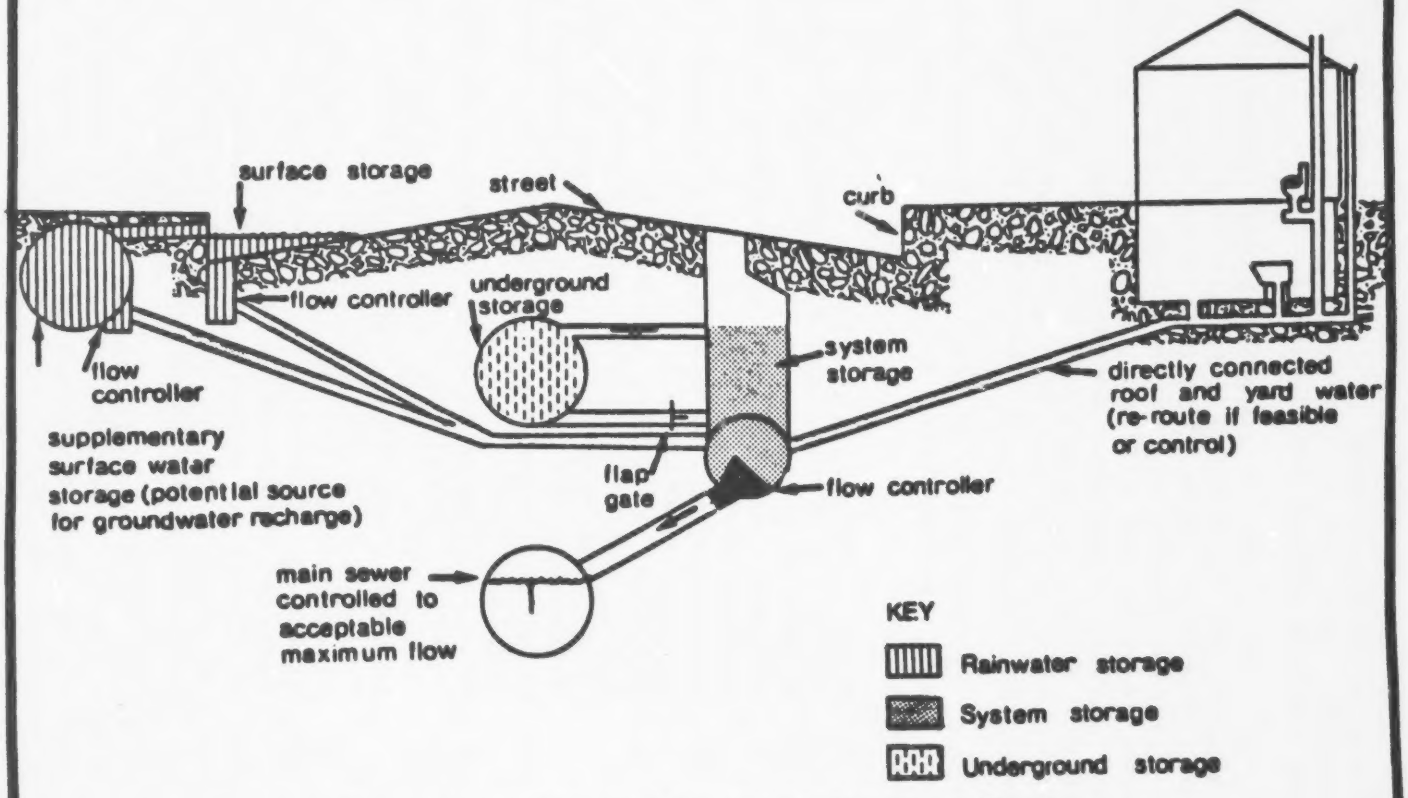

surface storage
street
curb
flow controller
underground storage
flow controller
system storage
directly connected roof and yard water (re-route if feasible or control)
supplementary surface water storage (potential source for groundwater recharge)
flap gate
flow controller
main sewer controlled to acceptable maximum flow
KEY
Rainwater storage
System storage
Underground storage

Figure 11.1 Components of the Inlet Control System

rerouting roof downspouts to splash pads with drainage to dry wells/created pervious areas/street catchbasins. One device used to augment this method is a "Eave-N-Flo" eavetrough regulator placed in the downspout. Soakways, dutch drains, porous pavement are other examples of potential inlet control methods used to increase perviousness/infiltration in urban areas.

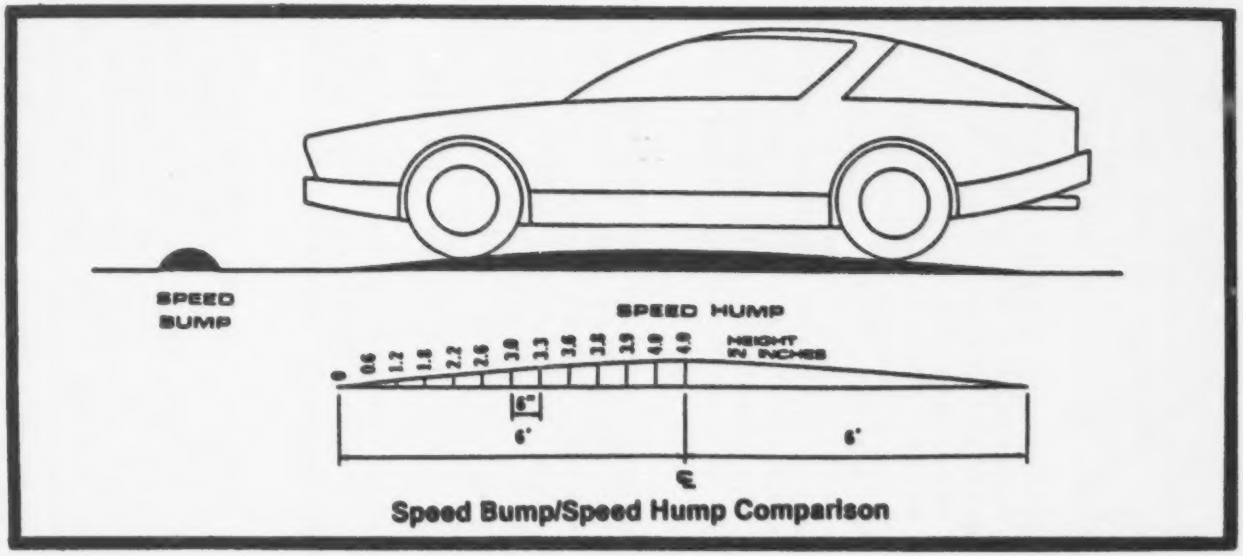
Temporary street ponding is still another method of "inlet control". Flow controllers can be placed in catchbasins to create temporary controlled street ponding. Pipes connecting catchbasins to the sewer are typically over-sized to prevent clogging from debris washed into catchbasins. Flow controllers either regulate flow into a sewage collection system or, after the water is in the system, they regulate the flow to a predetermined rate. Controllers placed at stormwater inlets control the flow of water into the combined sewer system, thus causing the water to pond in the streets, parking areas, etc. By preventing or slowing the overload, the sewage system is able to cope with the water, thus preventing overflows and basement backups.

Flow controllers can also be placed in catchbasins to strategically induce overland flow into areas more suitable for surface/subsurface storage. Speed humps (contoured berms not speed bumps) can be placed either to direct overland flow away from sensitive areas and into more suitable storage capture areas or to maximize street storage potential. Taken together catchbasin flow controllers and a system of speed humps can generate at low cost, controlled street storage.

The distinction between speed "bumps" and speed "humps" is shown in Figure 11.2 Speed bumps are typically not envisioned as these devices are a vehicular nuisance and interfere with snow plowing operations.

Placed at specific points within the system in-line flow controllers can also act as damming devices to utilize storage space that may be available in the existing sewers and/or direct flow to off-line storage facilities. Shallow underground/open off-line storage tanks/basins placed behind curbline/parkways/back alleys or in open park areas (see Figure 11.3) are other methods of "inlet control" which are meant to temporarily store excess stormwater beyond street storage for later controlled bleedback into sewer system. Shallow new storm drains connected into existing and/or new catchbasins and draining to storage tanks are often necessary to utilize underground storage.

The ability to accurately control the rates of flow entering the piped drainage system enables the effects of the


SPEED BUMP
SPEED HUMP
0
0.6
1.2
1.8
2.2
2.6
3.0
3.3
3.6
3.8
3.9
4.0
4.0
HEIGHT IN INCHES
6"
6'
6'
℄

Speed Bump/Speed Hump Comparison

FIGURE 11.2

Asphalt - Contoured Speed Hump

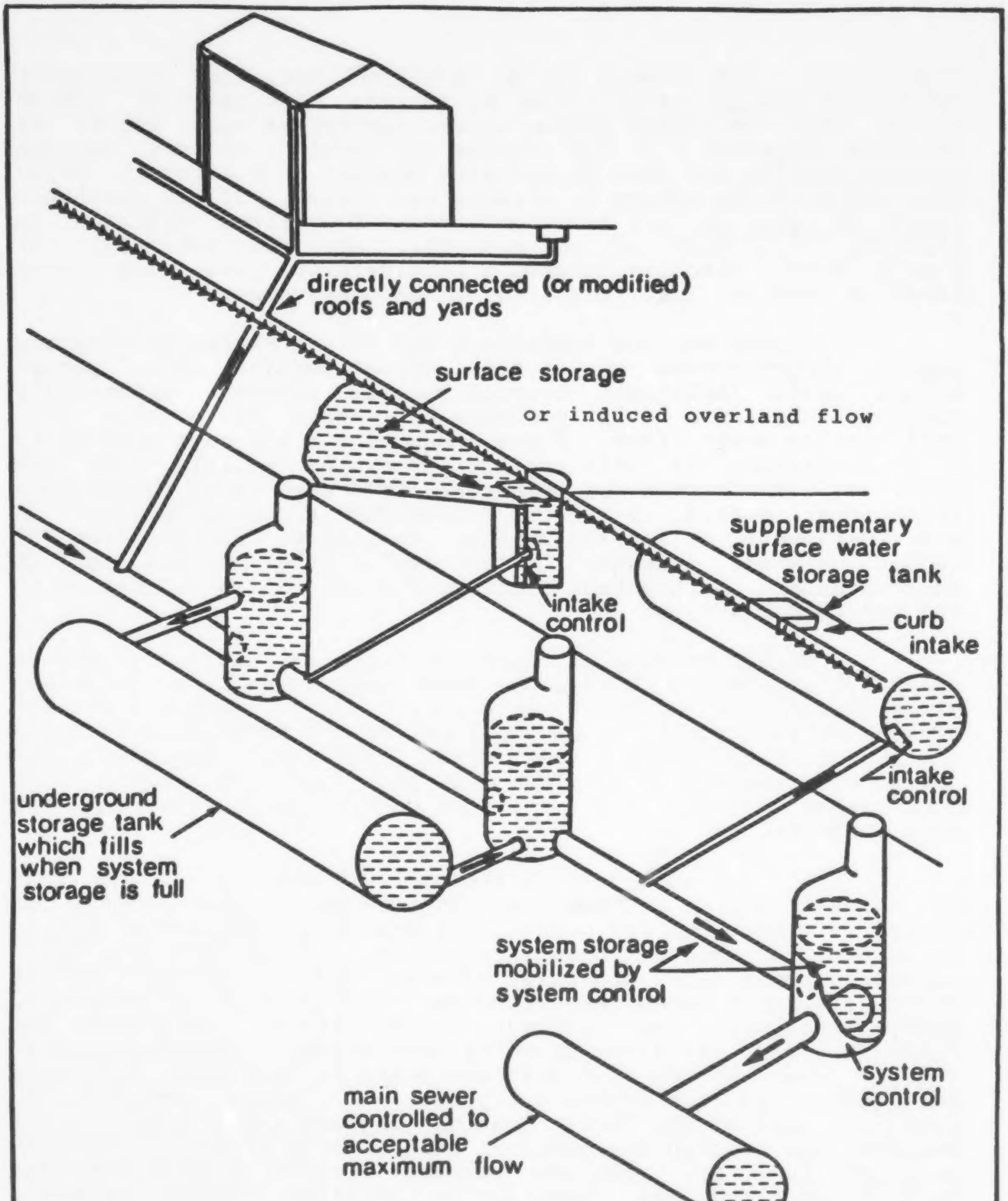

directly connected (or modified) roofs and yards
surface storage
or induced overland flow
supplementary surface water storage tank
intake control
curb intake
intake control
underground storage tank which fills when system storage is full
system storage mobilized by system control
system control
main sewer controlled to acceptable maximum flow

Figure 11.3 Surface / Subsurface Storage Components of Inlet Control for Congested Areas

inevitable major storms to be designed for, and consequent downstream damage and pollution minimized. When rates of inflow exceed the controlled system inputs any excess will either be retained adjacent to the intakes in local transient storage (street ponding and then in off-line storge), or enter the major overland drainage system to streams and rivers. If all available forms of storage are fully utilized and overland drainage is limited, restricted or not possible, then the potential for excess street flooding becomes a consideration requiring other forms of control, i.e., positive relief conveyance.

Street surface conditions and street surfacing programs impact effectiveness of potential street ponding since ponded waters could infiltrate through cracked pavement potentially causing subsurface soil problems and could drain back into infiltrating sewer lines. House foundation drain-back cycling is a consideration in this regard. Types of manhole lids are important. Replacement of perforated lids with solid covers is a traditional method of cutting inflow and is a consideration in the assessment of street ponding viability. Installation of vented neoprene inserts, "safeguards", below manhole lids is another method and has been successfully used in other cities in the USA to cut inflow.

Street ponding as a form of transient storage is always carefully engineered to meet specific local conditions. Draindown times are always a major concern as it impacts emergency vehicle access and is a public nuisance. Time for draindown is a function of the surface intake rates, sewer conveyance capacity (and outlet conditions) and overland flow potential. The singular most controlling factor is sewer system conveyance capacity and its outlet condition.

All of the above "inlet control" methods assume that the sewerage system and its appurtenances can in fact receive and convey a controlled and predictable amount of combined sewage to an interceptor system. Local systems are old and have many well-known problems. Part of "inlet control" must ultimately involve study of system soundness including, condition of catchbasins, sewer deposition and sediment accumulations, potential for siltation, and conveyance capacity limitations. Consideration of relief sewer segments in upstream areas to mitigate hydraulic bottlenecks is not beyond the purview of "inlet control". An integral part of the "inlet control" engineering analysis is to analyze and portray the physical and economic trade-offs between creating temporary upstream storage (street ponding/off-line facilities) and the constructing positive relief segments (systems) to accomplish the same effect.

Benefits of using the "inlet control" method in local areas with combined sewers can generally be summarized as: A) minimizes the need and degree for sewer separation and local relief sewers; B) provides immediate relief in areas with most severe basement flooding problems, leaving implementation in less critical areas when funds become available; C) lessens combined sewer overflows and pollution since retained transient stormwater (and combined sewer stored in-line) drain back to sewers for treatment; D) lessens local sewer deposition/reduced conveyance and decreases sewer cleaning frequency and costs; E) provides an extremely adaptive management approach for overall control that can be applied inexpensively and quickly in stages and then as time and funds permit, be integrated with positive relief measures to provide ultimate protection; and F) greatly enhances basement flooding protection levels for areas that have already provided local positive relief.

"Inlet Control" has a strong positive benefit for large complicated tunnel/lift station/conveyance/treatment systems. The net effect of applying "inlet control" to a number of catchment areas tied into a major system is to "base-load" the system output implying that flow peaks would be smaller and less frequent and that a higher system-wide base flow would result. Operational efficiency of any large hydraulic/electrical system always improves with "base-loading" system throughputs.

The "inlet control" approach can be immediately applied to provide initial benefits with balance of plan incrementally applied as funds become available. Community investment levels for "inlet control" can be relatively continuous in contrast to other approaches that are more "lumpy" in nature and in realizeable gains. Communities can expend and realize year by year incremental low level investments and their additive attendent benefits in contrast to a single "lumped" investment taking many years both to accomplish and to realize the benefit.

"Inlet control" can be readily added to areas where new local positive relief segments have been or are in process of being installed. The "inlet control" engineering analysis simply assumes that the existing sewer system supplimented with the new relief components/system now represents the target conveyance capacity for restricting inflows. Extremely high levels of basement flooding protection can be easily achieved. Furthermore, "inlet control" could be implemented along with the positive relief program and visibly add no greater inconvenience to local residents than would the positive relief system construction. Depending on the area and the potential for generating off-line storage, street flooding protection may slightly deteriorate. The exact trade-off has to be assessed on a case-by-case basis.

Amount of stormwater that can be reasonably and feasibly detained by "inlet control" methods is in part a function of the sewer system's capacity to convey flow. If the outlet conditions are unfavorable, i.e., backwater conditions prevail, then the full conveyance capacity of trunks and branch lines cannot be realized, hence shifting the stress for additional storage further upstream. If the outlet control for a given subsystem is adverse then the benefits of "inlet control" are obviously limited. "Inlet control" has the intrinsic benefit of enabling the designer to maximize the detentive storage potential of any system given its ability to convey flows. Rigorous application of "inlet control" philosophy does in fact reveal the true cost effectiveness of positive relief measures.

The commercial availability of a self-acting, liquid flow-limiting, non-clogging device has made possible the wide spread and cost-effective implementation of "inlet-control" drainage and pollution control solutions. Stated simply, the maintenance-free vortex valve technology detailed in Chapter 5 permits solutions entailing multitudes of "micro" level flow controls in a given system. Without this technology controlling hundreds (or perhaps thousands) of inlets in a given sewerage system would be extremely difficult. Prior to the advent of vortex valves "inlet control" philosophy for controlling adverse flow situations was limited.

11.3 Overview of Hydrologic Methodology

Hydrologic analysis within a given study area is accomplished in the following stages of work. The watershed is subdivided into logical subcatchments. Subareas are defined on the basis of topographic divides and on sewer subsystem configuration. Overall imperviousness per catchment is computed using aerial maps. Directly connnect areas such as residential and commercial roof areas are estimated and tabulated per subcatchment. The number of residential and commercial buildings is tallied per subcatchment.

Hydraulic analysis of the drainage system within the study area is performed to determine conveyance capacity of each manhole to manhole segment throughout the system and per subcatchment area. Identification of key drainage pipe segments and their respective full-pipe or allowable conveyance capacities per catchment forms the basis to compute total required storage (system, surface and underground) for a given design storm per subcatchment. Allowable capacity is defined as some flow rate less than the segment capacity. The additive capacity of some lateral segments can exceed the capacity of the main trunk line. Prevention of adverse surcharge of the trunk line sometimes requires that the lateral line capacities be artificially lowered to a selected allowable rate of flow.

Design of rainfall storm intensities for the 5, 10, 15 and 25 year events is extracted from established rainfall intensity/duration curves. A computer model using the Unit Hydrograph approach is then used to analyze all subcatchments for each design storm with and without roof-top directly connected areas. Design curves are established for each subcatchment per design storm relating total volume of required storage to the rate of subcatchment outlet discharge (full-pipe or allowable pipe flow conditions). These curves are then used to compute total required storage with/without downspout control per subcatchment per design storm.

Total required storage per design storm for full pipe outlet discharge conditions (or allowable discharge) are then compared with available storage computed for each subcatchment. Available storage equals the sum of system storage (pipe volumes plus manholes/catchbasins) and allowable surface storage. Storage deficits (total required - available) are treated either by direct underground storage or by allowing overland flow. Controls are prepared for a given subcatchment and for a given design storm. If the sum of the uncontrolled peak roof discharge plus controlled street catchbasin discharge do not exceed full pipe or allowable subcatchment outlet conditions, then no additional control is necessary. If outlet flow or allowable conditions cannot be met, then the downspouts in a given subcatchment are controlled for a given design storm such that the sum of the controlled roof peak plus controlled catchbasin flow plus release rates from underground storage tanks, if necessary, approximately equals the full pipe or allowable subcatchment conveyance capacity.

Draindown times from the new underground storage tanks (a function of release discharge rates) is a critical concern in the analysis. This problem is further compounded by the need to control household roof downspouts to a reasonable and feasible extent.

Control requirements within a subarea are a function of the hydrologic, topographic and sewer system characteristics of the subarea and the effect of the subarea sewer system and overland flows on downstream areas. As described above, allowable flows are sometimes utilized rather than capacities. Allowable flow rates are adjusted where possible to ensure that the degree of control required is reasonably homogeneous throughout the area. Surface storage usage is maximized in all cases with overland flow being promoted wherever possible in lieu of underground storage to minimize the number of underground storage locations. Underground storage tanks are utilized to detain street runoff from catchbasins without flow controllers.

Sewer Hydraulic Analysis

The conveyance capacity of the sewer system within the study area is analyzed using a computerized network model of each segment (manhole to manhole). Pipe length, diameter, and pipe slope for each segment are tabulated and data-processed into a network system to determine maximum full pipe flow. The purpose of this investigation is to compute the maximum stormwater flow allowable in each segment.

Maximum flow capacities per segment are then used in two further investigations. First, they are compared segment to segment to ensure that each downstream segment is of sufficient capacity to handle upstream flow. Next, the segment downstream of each pipe junction is reviewed to ensure it can pass the combined capacity of upstream branches.

Control schemes for each subarea are then devised to ensure adequate protection in each area and adjusted as possible to utilize consistent control measures among sub areas.

Subarea control plans are formulated as follows. Subareas are reviewed to indicate the "allowable" flow, the degree of catchbasin and downspout control required to achieve that flow, required storage at that flow, available storage (system plus surface) and the net storage deficit. Storage deficit can then be met using underground storage or overland flow. Minimizing the number of underground storage sites is always considered advantageous. Routing of overland flow to common collection points is always preferred. One limitation to overland flow routing is the street gutter capacity to pass accumulated flows without curb topping.

Downspout Regulation

Directly connected roof downspout discharges can exceed full pipe conveyance capacity. In some cases, no amount of surface/subsurface storage will eliminate this problem. It is recognized that control of downspouts is considered undesireable. All other forms of control are maximized to the extent considered feasible and practical before downspout control is considered. The aim of this strategy is to minimize the extent and degree of downspout regulation. Furthermore, every effort is typically made in the analysis to develop as reasonably uniform policies as possible per area per design storm.

Two different approaches are possible for roofwater control analysis and include,

a) disconnect both front of the house downspouts with placement of splash pads,
b) disconnect both of front of house downspouts and place Eave-N-Flo regulators in rear of the house downspouts.

Each subcatchment area is analyzed to ascertain what degree of roofwater control is necessary to ensure that the resulting roofwater discharge together with catchbasin controlled flow and allowance for underground storage tank discharges would not exceed sewerage system full pipe discharge.

Surface Storage Improvements

The concept of surface storage of stormwater requires that the natural overland routes for surface water be altered to create storage areas. The degree to which these alternatives can be accomplished without excessive citizen inconvenience dictates the overall effectiveness of this flood protection technique. Two typical criteria established to maximize storage and minimize inconveniences are that storage berms (speed-humps) constructed across roadways be spaced a minimum of 300 feet apart and surface ponding not exceed in the curb height.

Berms are constructed of asphalt with an elevation equal to the elevation of the curb. The berm at the street edges would equal the curve elevation. As the street elevation increases toward the crown the effective berm height would lessen (elevation constant). This implies that storage berm heights would vary across the street based on curb heights and street contours. Construction of storage berms would recognize this effect by adjusting the length of the berm to minimize the "speed bump" effect. Based on these criteria the effective storage capacity of any street section is a function of the street slope, street cross section and curb heights.

Two types of street storage are available. The first utilizes existing low areas in the street profile. A maximum desirable depth of ponding is first determined based on available curb heights and acceptable center street depths at the lowest point in the center of street. Street slopes in each direction are then used to determine the distances to which water will pond. Ponded water volume is calculated as a function of this distance and the average cross- sectional area. Volume calculations for ponding in low areas utilizes the backwatered distances in each direction. It is necessary that catchbasins exist or be constructed in the low area to act as continuous drains.

The second type of street ponding utilizes the berm construction (speed-hump) technique to reduce surface runoff on mildly sloped streets. The effectiveness depends substantially on the slope of the street. Berm site location is typically just downhill of existing catchbasins to allow final drainage. Storage volume is calculated in a similar manner as above using curb height, street contour and slope data.

Several criteria are used in the final selection of proposed storage sites. First, storage capacity of low areas, where the street slopes uphill in each direction, is calculated on the basis of allowing the stored water to pond to a level up to the street side of the sidewalks. Storage capacity of slightly sloped areas is calculated as surface waters being ponded to the curb line. Second, it is typically assumed that berm construction is cost effective only if the volume of storage is provided at a unit cost under $9/cu.ft.

11.4 Saginaw Sewerage System

Reported occurrences of basement flooding in many areas within the City of Saginaw have been recorded for many years. The City has somewhat reduced the problem by simply installing orifice plates in numerous catchbasins in an effort to reduce inflow. This effort represents a limited program of inlet control planning and implementation. Figure 11.4 illustrates areas within the City where basement flooding problems have occurred and the catchbasin orifice-plate solution attempted. Success of this approach has been mixed in Saginaw somewhat due to the reported practice of residents removing the orifice plates.

As part of the sewerage system evaluation of this study, upstream areas of basement flooding were reviewed. A City supplied base map indicated the historical problematic areas. (Figure 11.4) Twenty seven areas were identified within the City to have reported one or more occurrences of basement flooding. Sizes of areas ranged from individual households to several acres with a total problem area of roughly 455 acres. Problem areas are scattered throughout the City.

Each area was visually inspected by EDP personnel to determine land topography and usage and other characteristics. In general, it was found that most areas were relatively flat, densely populated and afforded little open space. The significance of open space is its potential usage as an

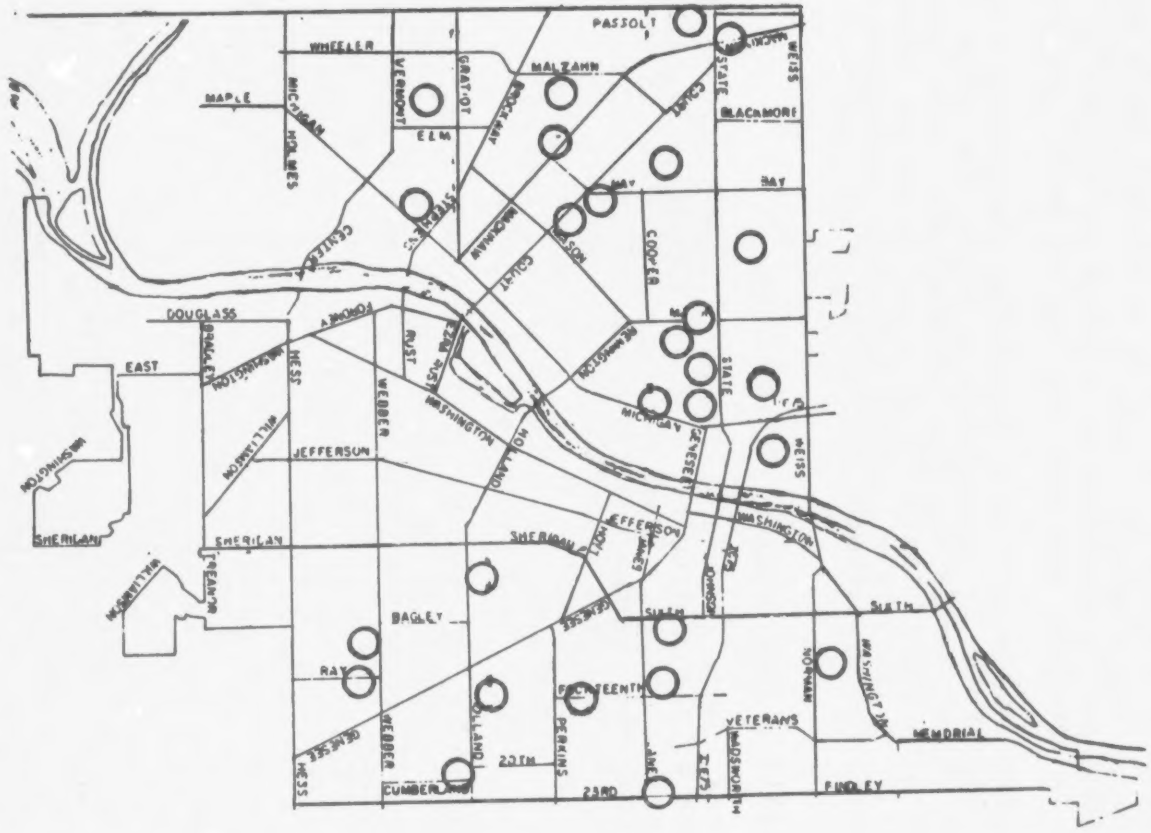

PASSOLT
WEISS
STATE
BLACKMORE
BAY
WHEELER
MALZAHN
VERMONT
GRATIOT
ELM
MAPLE
MICHIGAN
HOLMES
BROCKWAY
MACKINAW
COURT
COOPER
CENTER
STEPHENS
DOUGLASS
EAST
BRADLEY
WASHINGTON
HESS
WILLIAMSON
WEBBER
RUST
EZRA RUST
REMINGTON
MICHIGAN
GENESEE
JEFFERSON
HOLLAND
SHERIDAN
TREANOR
JOHNSON
I-675
HOYT
JANES
BAGLEY
RAY
SIXTEENTH
PERKINS
20TH
23RD
CUMBERLAND
VETERANS
WADSWORTH
NORMAN
MEMORIAL
FINDLEY
SIXTH

Figure 11.4 City of Saginaw areas of Reported Basement Flooding

underground storage location. Although streets and sidewalk areas are useable, construction costs in open areas are less. In general, areas of Saginaw where little or no open space exists do include some sidewalk spaces without trees. Underground storage chamber placement is preferred in sidewalk as opposed to street areas, but is sometimes not practical due to tree-lined streets.

City records were available indicating basement flooding complaints by households up until the late 1970's. Since that time, complaints have lessened and gone unrecorded. One problem area was isolated for detailed review as reported in Section 11.5 to illustrate typical evaluation techniques and to form the basis for system-wide extrapolation of results.

11.5 Analysis of the Congress Courts Area

One isolated area in the northwestern part of the City has been the source of frequent basement flooding complaints and was reviewed in detail. This area contains about 16 acres and includes Congress Avenue between Brenner Street and Bay Street and the four Congress Courts, (A,B,C,and D). (Figure 11.5) Each court is approximately 300 ft long extending from Congress Avenue to a cul-de-sac. The courts are serviced by a combined sewer system with 12 inch pipes in each street.

As part of the detailed analysis each street was completely surveyed and a contour map of the area developed. Figure 11.6 illustrates a portion of the contour map including one court which is typical of the surface topography in Congress Courts A, B and C. Each of these streets slopes downhill from Congress Avenue to the start of the cul-de-sac and rises thereafter. Catchbasins are located on each side of each street in the lowest area at the cul-de sac and at Congress Avenue. Congress Court D slopes downhill from the cul-de-sac toward Congress Avenue the entire length of the street. Directions of overland flow are indicated on Figures 11.5 and 11.6. For the purpose of computer-aided evaluation, each court was designated as a separate drainage area. Hydrologic and hydraulic analyses were then performed according to the procedures outlined in Section 11.3. Standard evaluation procedures would have included detailed analysis of surface storage potential based on area topography and street curb elevations. For the purpose of this illustration a nominal value of 300 cubic feet per acre (24.4m3/Ha) (typical of this type topography) was utilized in Table 11.1.

Table 11.1 depicts a standard work sheet utilized by EDP for subarea evauations and Table 11.2 includes the computerized hydraulic analysis of the area sewer network. Table

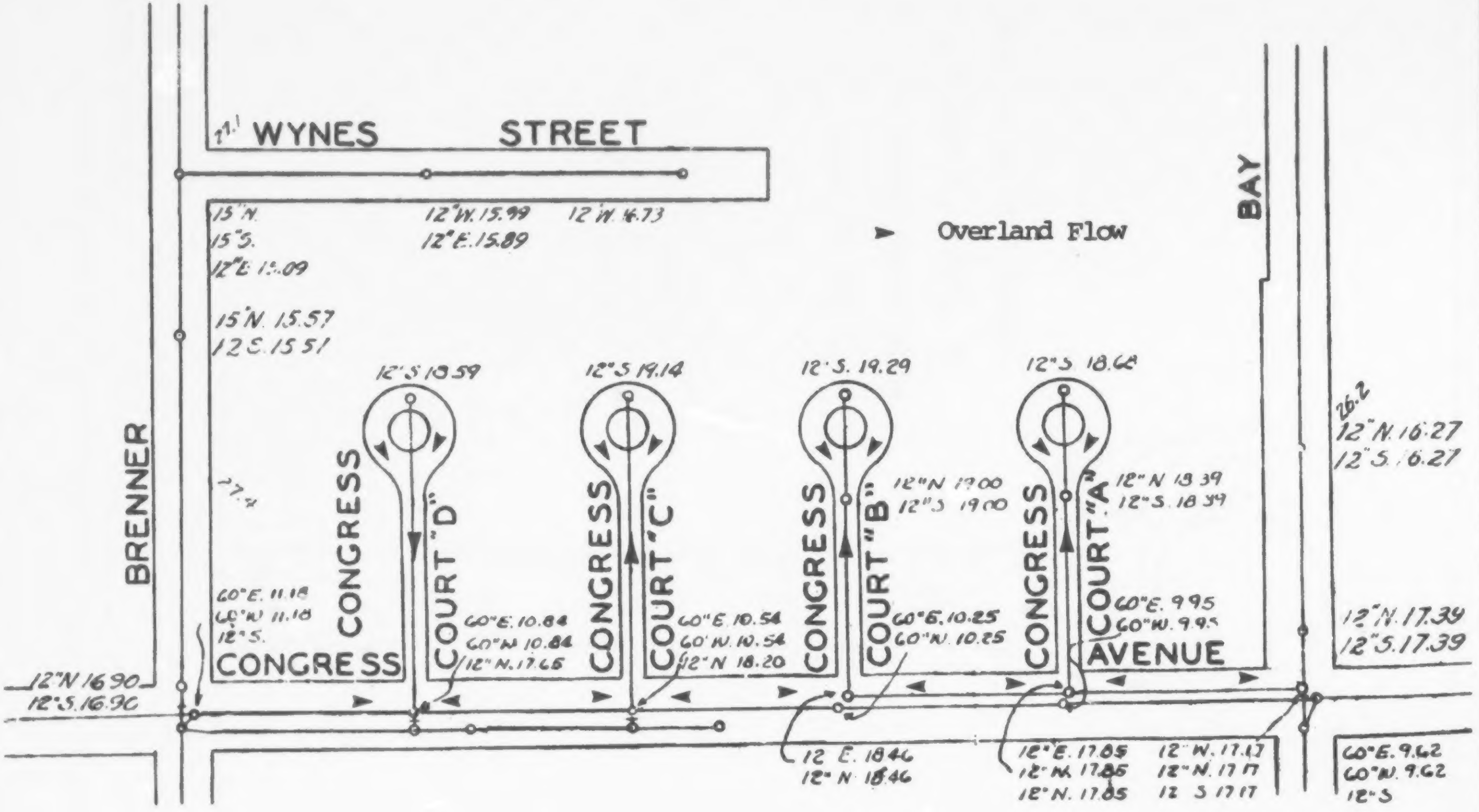

27.1
WYNES
STREET
15"N.
15"S.
12"E 15.09
12"W.15.99
12"E.15.89
12"W 16.73
Overland Flow
BAY
15"N. 15.57
12"S. 15.51
12"S 18.59
12"S 19.14
12"S. 19.29
12"S 18.62
26.2
12"N. 16.27
12"S. 16.27
BRENNER
27.4
CONGRESS
COURT "D"
CONGRESS
COURT "C"
CONGRESS
COURT "B"
12"N 19.00
12"S 19.00
CONGRESS
COURT "A"
12"N 18.39
12"S. 18.39
60"E. 11.18
60"W 11.18
12"S.
CONGRESS
60"E. 10.84
60"W 10.84
12"N. 17.65
60"E. 10.54
60"W. 10.54
12"N 18.20
60"E. 10.25
60"W. 10.25
60"E. 9.95
60"W. 9.95
AVENUE
12"N. 17.39
12"S. 17.39
12"N 16.90
12"S. 16.90
12 E. 18.46
12" N 18.46
12"E. 17.85
12"W. 17.85
12"N. 17.85
12"W. 17.17
12"N. 17.17
12 S 17.17
60"E. 9.62
60"W. 9.62
12"S

Figure 11.5 City of Saginaw Inlet Control Study Areas - Congress Court

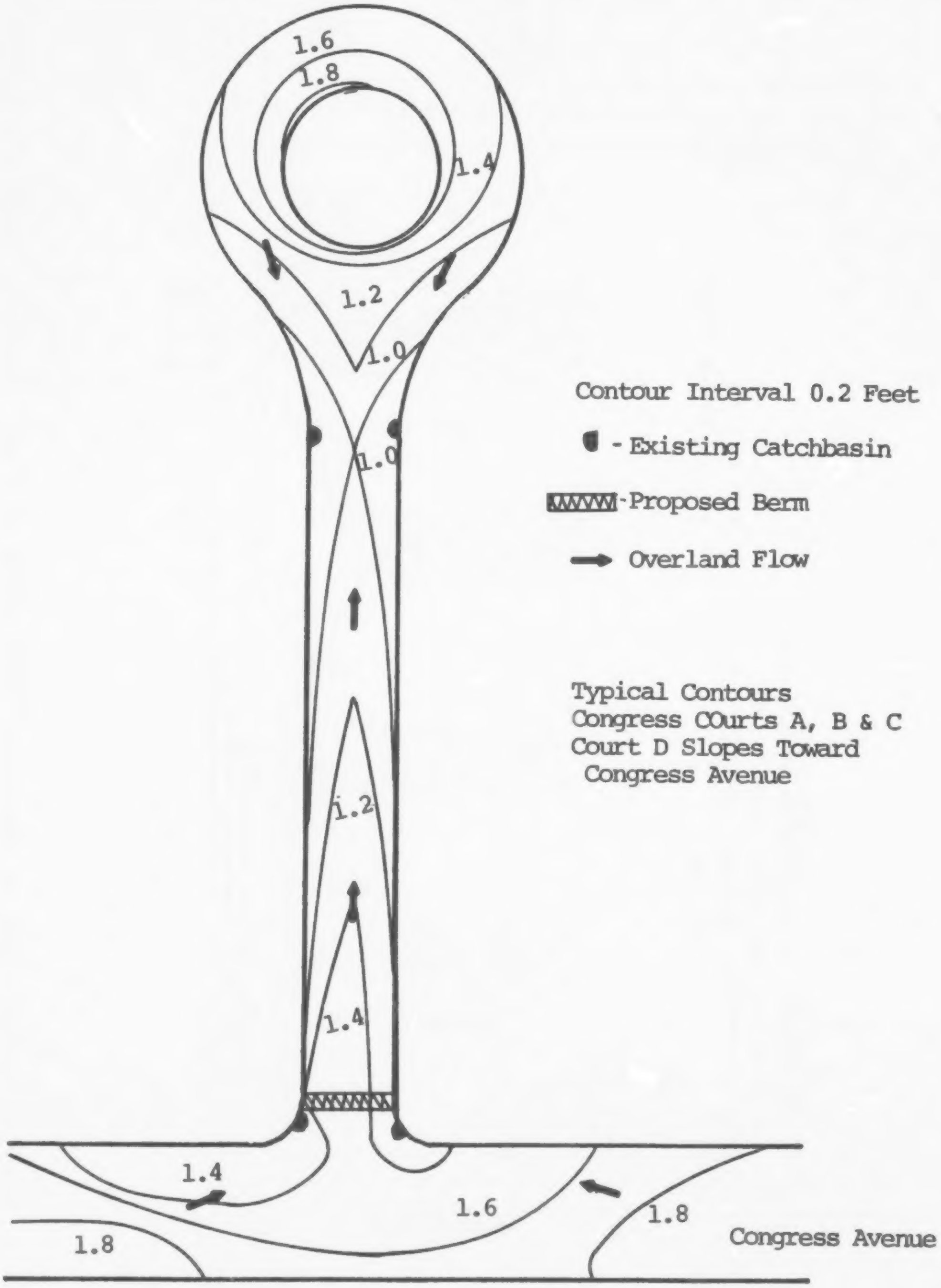

1.6
1.8
1.4
1.2
1.0
1.0
Contour Interval 0.2 Feet
- Existing Catchbasin
-Proposed Berm
Overland Flow
Typical Contours
Congress COurts A, B & C
Court D Slopes Toward
Congress Avenue
1.2
1.4
1.4
1.6
1.8
1.8
Congress Avenue

Figure 11.6 Typical Congress Court Topography

11.2 was developed based on manhole to manhole pipe segment lengths and slopes generated from Figure 11.5. Individual segment capacities, cumulative flows rates and other statistics are then calculated within Table 11.2 as required for completion of the worksheet in Table 11.1. In addition, a rainfall/runoff and storage requirement relationship is developed. Table 11.3 illustrates a typical computer printout indicating the relationship of allowable subarea outflow rates to required volumes requirement of storage. For the Congress Court subareas, the 12 inch pipe capacity of 49 l/s (Table 11.2) necessitates a total storage volume of 86 cubic meters for the 10 year frequency storm event as found by interpolation within Table 11.3. (Downstream restrictions to Congress Courts A and B outflow are discussed later.)

The concept of the remaining calculations is to select an appropriate combination of roof stormwater control, catchbasin inlet rates, surface storage draindown rates, and underground tank draindown rates, which will minimize post storm surface ponding, minimize underground tank storage requirements and draindown times while still maintaining total outflow rates within allowable levels.

Table 11.1 indicates that available storage would include surface storage of 24.4 m3 and system storage of 7.9 m3 for a total of 33 m3. For the 2 year design storm, a storage tank volume of 23.3 m3 (845 ft3) is required and would drain in 1 hour which is generally acceptable. No roof control is requird. Two of the four catchbasins (those located at Congress Avenue) would be restricted with vortex valves at an outflow rate of 3 l/s. The two catchbasins at the cul-de-sac would be uncontrolled and directly connected to the storage tank.

Designs for the 5 and 10 year storms included similar control except each design required that roof front downspouts be disconnected and larger storage tanks be provided. Control of roof discharges allows for increased tank discharge rates and results in draindown times less than that designed for the 2 year event. Design for the 25 year storm resulted in a lengthy storage tank draindown time of 130 minutes for the case of front roof control. Additional control of rear downspouts would allow greater tank discharge rates and substantially reduce the draindown time. In general, the inlet control strategy is often a case of such alternatives. Here the choice is between the extent of roof runoff control and tank draindown time. It is at this point where engineering judgement is required to select the best combination of options.

Although straightforward, inlet control design is often a complicated series of analyses. For the Congress Courts area

TABLE 11.1

TYPICAL INLET CONTROL WORKSHEET

Congress Courts Saginaw, Michigan

Description of Subarea Courts A,B,C 1.16 Ha

Impervious Area Ha: .464 Roof Area: .11 % Roof Area: 24

No. of Houses: ~10 NO. of Commercial Buildings: 0

No. Catchbasins: 4 No. Inlets: 0

Time of Concentration(Min.) : 7.5

Capacity Full Pipe l/s : 49 Half Full Pipe l/s: -

System Storage Full Pipe m^3 : 7.9 Half Full Pipe m^3 : -

Surface Storage m^3 : 24.4

		Design Storm			
		2 Year	5 Year	10 Year	2 Year
Average Rainfall Intensity in/hr mm/hr		74	92	105	124
Peak Discharge l/s					
	Total	154	191	218	258
	Street	117	145	166	196
Total	R1 Roof	37	46	52	62
Front Disconnected	R2 Roof	18	23	26	31
Rear Restricted	R3 Roof	13	13	13	13
Surface Storage as Gutter Flow m^3					

Design Storm	Flow l/s	Roof Control l/s	CB Rate * l/s	Req. m^3	Storage Avail. m^3	Need m^3	Surface Storage Disch. l/s	Tank Storage Disch. l/s	Drain Time
2	49	R1 37	2@3=6	56		23	-	6	66 min
5	49	R2 23	6	75		42	-	20	36 min
10	49	R2 26	6	86		53	-	17	53 min
25	49	R2 31	6	106		73	-	12	103 min
25 alt.		R3 13	6	106		73	-	30	41 min

* Note: 2 of 4 (Total) CB lead to tank storage

TABLE 11.2 City of Saginaw Congress Courts Sewer System Hydraulic Evaluation

STORM DRAIN DATA FOR CONGRESS COURTS

NO	LOCATION	CODE	DIA(M)	SLOPE	LENGTH	F. FLOW(L/S)	F. DEVIATION	VEL.(M/S)	TRAVEL (MIN)	PIPE VOL.(CU M)	HEAD LOSS(M)	TOTAL LOSS(M)
1	COURTB	1.01	0.305	0.23	38.10	48.57	0.00	0.67	1.0	2.78	0.09	0.09
2	COURTB	1.02	0.305	0.24	70.10	48.88	0.31	0.67	1.7	5.12	0.16	0.25
3	CONGRESS	1.03	0.305	0.24	79.20	48.88	0.00	0.67	2.0	5.79	0.19	0.44
4	COURTA	2.01	0.305	0.23	38.10	48.57	0.00	0.67	1.0	2.78	0.09	0.09
5	COURTA	2.02	0.305	0.24	70.10	48.88	0.31	0.67	1.7	5.12	0.16	0.25
6	CONGRESS	1.04	0.305	0.25	83.80	50.11	-47.65	0.69	2.0	6.12	0.21	0.65
7	BAY	1.05	0.305	0.32	21.30	57.04	-90.84	0.78	0.5	1.56	0.07	0.71
8	BAY	1.06	0.305	0.32	64.00	57.04	0.00	0.78	1.4	4.68	0.20	0.92

STORM DRAIN DATA FOR CONGRESS COURT 2

NO	LOCATION	CODE	DIA(M)	SLOPE	LENGTH	F. FLOW(L/S)	F. DEVIATION	VEL.(M/S)	TRAVEL (MIN)	PIPE VOL.(CU M)	HEAD LOSS(M)	TOTAL LOSS(M)
1	CONGRESS	1 01	1.520	0.14	76.20	2694.65	0.00	1.49	0.9	138.27	0.10	0.10
2	COURTD	1 .01	0.305	0.29	99.10	54.30	54.30	0.75	2.2	7.24	0.29	0.39
3	CONGRESS	1 .02	1.520	0.12	77.70	2510.00	2455.70	1.39	0.9	140.99	0.09	0.48
4	COURTD	12.01	0.305	0.29	99.10	54.30	0.00	0.75	2.2	7.24	0.29	0.29
5	CONGRESS	1 .03	1.520	0.12	73.20	2531.18	21.18	1.40	0.9	132.83	0.09	0.57
6	CONGRESS	1 .04	1.520	0.12	79.20	2477.89	-2563.30	1.37	1.0	143.72	0.09	0.66
7	CONGRESS	1 .05	1.520	0.11	88.40	2467.09	-10.80	1.36	1.1	160.41	0.10	0.76

TABLE 11.3 City of Saginaw Congress Courts Rainfall/Storage Requirement Summary

```
                 SUMMARY OF STORM HYDROGRAPH
                             FOR
DETERMINATION OF REQUIRED STORAGE AT DESIGN FLOW BASED ON
SYSTEM CAPACITY AND THE STORM/AREA CHARACTERISTICS
-----------------------------------------------
JOB--SAGINAW   CONGRESS AVE AREA
     ------------------------------------------
CASE--H   10 YEAR
      -----------------------------------------
TIME OF CONCENTRATION (MINS)= 7.5
TOTAL IMPERMEABLE AREA (HA) = .4640001
MEAN RAINFALL INTENSITY (MM/HR) = 105
RAINFALL CLASS (%) = 50
-----------------------------------------------
MAX. RATE OF STORM FLOW (L/S) = 218.4
-----------------------------------------------
VOLUMES OF STORM FLOW RELATED TO
DIFFERENT OUTFLOW RATES
-----------------------------------------------
RATE OF OUTFLOW         VOLUME OF STORMWATER
   (L/S)                      (CU.M.)

    0                          108
    21                         98
    43                         89
49- 65                     86- 81
    87                         73
    109                        66
    131                        59
    152                        53
    174                        46
    196                        37
    218                        21
-----------------------------------------------
```

the inlet control plus storage option detailed above must be compared to a relief sewer system on feasibility, cost and consequence basis. The relief sewer option is dependant upon sufficient capacity at some downstream location to pass all upstream flows.

Figure 11.5 indicates that a 60 inch sewer serves Congress Avenue. Field monitoring and/or upstream and downstream evaluations can be performed to determine if sufficient capacity is available to carry fully unrestricted stormflow from the Congress Courts. If so, cost analysis should be performed comparing storage tank construction to that of a relief sewer system. As an example, 200 ft. of 18 inch relief sewer at a cost of $120/ft would require $24,000 to relieve one court area. At a cost of $9/ft3, 2670 ft3 (75m3) of surface storage could be constructed for the same sum. On this basis, and reviewing the storage requirements of Table 11.1, design for the 25 year event should consider a relief system as an alternative. As detailed in Section 11.6, inlet control plus storage will afford some downstream CSO reduction and this aspect must be considered as part of the overall engineering evaluation.

Hydraulic analysis of the existing Congress Courts sewer system indicates additional complications within the area. As seen in Figure 11.5 sewer lines servicing Congress Courts C and D discharge directly to the 60 inch Congress Avenue trunk line at drop manholes. Unless extreme surcharge exists in the 60 inch line, then free discharge from Congress Courts C and D is expected. Sewers servicing Congress Courts A and B do not discharge to the 60 inch line but join at a 12 inch sewer in Congress Avenue leading to another 12 inch sewer in Bay Street. Table 11.2 indicates that this junction represents a flow restriction based on accumulated flows.

One optional solution would entail designating "allowable" flow rates in each upstream sewer. Considering the downstream capacity at the junction of 50 l/s each, the upstream sewer could be "allowed" a rate of 25 l/s. Calculations similar to the worksheet of Table 11.1 and data from Table 11.3 for an outflow rate of 25 l/s indicate that even for the 2-year event, complete roof runoff control and storage of about 96 m3 would be required. Clearly an alternative would be desireable.

The most apparent viable alternative would be connection of the sewer from Court B to the 60 inch Congress Avenue sewer. This would allow the full capacity of the Court A sewer to flow to the 12 inch Congress Avenue sewer. This alternative is again dependent upon conditions within the 60 inch sewer. However, considering the degree of restriction and storage required under the "allowable" 25 l/s flow rate it would appear preferable to "create" capacity within the 60 inch line by implementation of inlet control in other upstream and downstream areas as necessary. This alternative would lead to the storage requirements detailed in Table 11.1. As stated earlier, Congress Court D slopes toward Congress Avenue the entire length. If the 60 inch Congress Avenue sewer can handle the outflow from this area, then only suitable catchbasin and connector piping is required. If the 60 inch pipe capacity allowance is limited, then underground storage will be necessitated at this location.

In sum, the analysis of the Congress Courts areas demonstrates the complexity of inlet control/storage technology planning. The need to evaluate hydraulic conditions of receiving sewers dictates that even analysis of a small subarea will require some degree of overall area review. The key to cost effective inlet control technology is to not overly restrict one subarea in relation to another or to allow excessive outflow from one subarea at the expense of the alllowable outflow rate from another subarea. Outflow rates should be selected both on the basis of existing hydrualic capacities and the effects of increased/decreased rates on overall area hydraulics.

11.6 City Wide Implementation of Inlet Control Technology

EDP's city-wide visual review of the storm related problem areas of Saginaw indicated that these areas are quite similar to areas of other cities where similar problems existed, were evaluated and solved using inlet control technology. The detailed review of the Congress Courts area indicates that solutions used in other cities including downspout control, catchbasin control, storage, repiping and construction of relief sewers are applicable to Saginaw.

Optimum implementation of inlet control technology requires that the entire drainage system be evaluated to ensure appropriate allotment of trunk line capacity to subareas. Based on previous inlet control studies in areas similar to Saginaw, it is estimated that available surface storage which can be feasibly generated, amounts to 300 ft3/acre. Analysis of the Congress Courts area indicated that mixed usage of underground storage and added relief sewers or short connections is optimal. Although underground storage requirements are related to sewer capacities in addition to topographic conditions, most sewerage systems, on the average, are similarly designed and provide similar capacities. On this basis, underground storage requirement of 1000 ft3/acre (typical of previous inlet control study results for 5 year to 10 year design storm protection) is reasonably applicable and agrees well with the 10 year storm requirement (900 ft3/acre) for Congress Courts.

For the 455 acre area of Saginaw reportedly subject to basement flooding the potential available surface storage is estimated as 136,500 ft3 while a total of 455,000 ft3 of underground storage can be feasibly generated. Creation of storage areas using the methods detailed in section 11.2 may be a cost effective alternative in comparison to construction of a relief sewer systems and has the added advantage of reducing peak flow conditions in downstream areas. In Saginaw peak flow conditions often translate into CSO and related Saginaw River pollution.

Creation of upstream off-line storage under the program of inlet control affects downstream conditions essentially the same way as implementation of upstream in-line storage. Much of the BMP plan development of this study evaluated the effects of upstream in-line storage on downstream CSO conditions. This data can be used to predict the effects of inlet control strategies.

Since EPA has placed much emphasis on the reduction of phosphorous in the evaluation of CSO impacts, reductions of total phosphorous loading to the Saginaw River were therefore used to evaluate the impact of potentially solving the purported basement flooding in the 455 acres using stormwater management techniques. Figure 11.7 illustrates the percentage reduction of total phosphorous loading to the Saginaw River as related to in-system storage volumes. Zero reduction would imply complete overflow to the river with no treatment at the WWTP. Storage values represented volumes generated at regulator chambers and in upstream acres. At the present time the reduced BMP plan has been completely implemented. Storage volumes estimated under inlet control strategies were added to the reduced BMP plan storage volumes to represent predicted conditions. Two curves are shown in Figure 11.7. The solid line represents percent system wide wet weather phosphorous reduction versus storage volume based on data for pre BMP conditions, reduced BMP conditions and complete BMP conditions simulations detailed in Chapter 10 and included in Table 10.7.

On the basis of this curve (Figure 11.7) total phosphorous reduction would equal 29% after implementation of the surface storage provisions (300 ft3/acre) of inlet control technology and construction of underground storage facilities (1000 ft.3/acre). This values is 4.5% above the current post reduced BMP level of 24.5% (Table 10.7).

A substantial portion of the CSO pollution reduction attributable to the reduced BMP plan was due to increased flow rates at the Weiss Street crossover junction. The dashed line in Figure 11.7 represents a revised relationship of total phosphorous reduction to storage volume without including effects

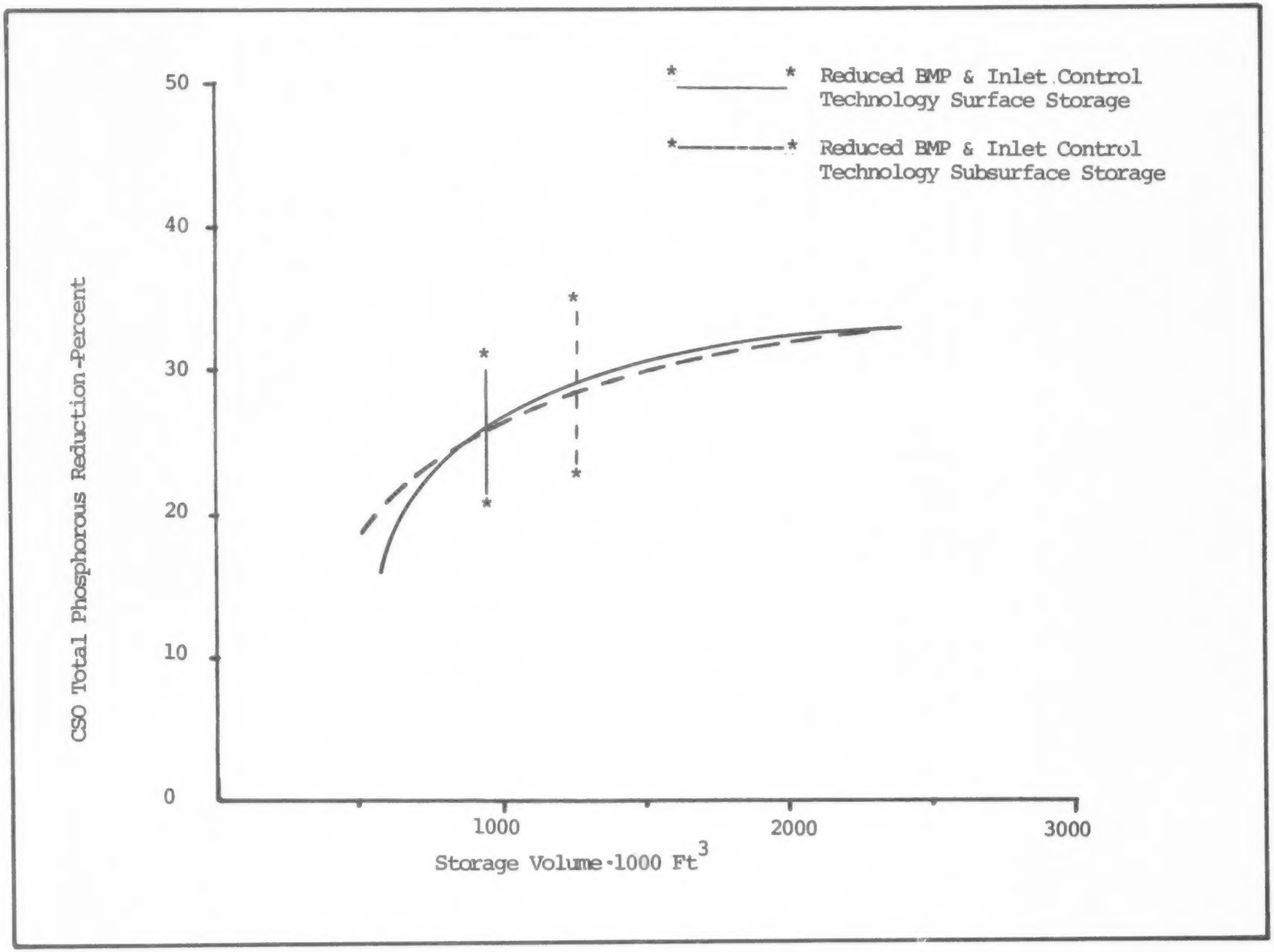


Figure 11.7 Relationship of Storage Volume to CSO Phosphorous Reduction

of the increased crossover flow. The inlet control scheme is estimated to reduce loadings to the Saginaw River by 4.0% above current post reduced BMP plan levels.

In sum, inlet control technology may be a cost effective alernative to relief sewer construction in the Saginaw area. Inlet control combined with provisions for adequate storage will mitigate basement flooding problems while reducing CSO pollution to the Saginaw River.

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